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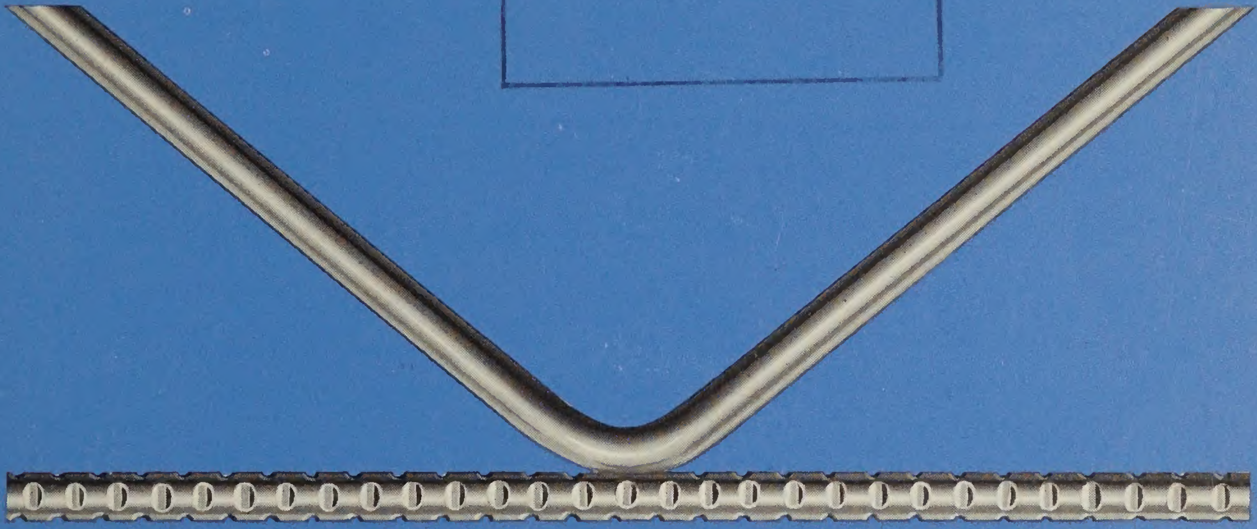
MAY 1963



DUR-O-WAL®

The Masonry Wall Reinforcement with Truss Design

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A Comprehensive Data File on Masonry Wall Reinforcement
compiled by W. H. BARTLETT, P. E., Chief Engineer for Dur-O-waL

MANUFACTURERS: CEDAR RAPIDS, IA. • SYRACUSE, N.Y. • PHOENIX, ARIZ. • BIRMINGHAM, ALA. • TOLEDO, OHIO • AURORA, ILL. • BALTIMORE, MD. • PUEBLO, COLO. • MINNEAPOLIS, MINN. • SEATTLE, WASH. • HAMILTON, ONTARIO, CANADA

FOREWORD

We have endeavored in this Comprehensive Data File to draw together all the major recommendations on masonry wall reinforcing proposed, or adopted by agencies of the Federal Government, private research organizations, and, of course, The Dur-O-waL Company.

This work is as complete, factual, and objective as possible. The File contains conclusions to which we do not fully subscribe as a company, but we well realize that some facets of masonry wall reinforcing still require further research and development.

We submit this Comprehensive Data File with an invitation for your comments.

Cedar Rapids, Iowa, July, 1962

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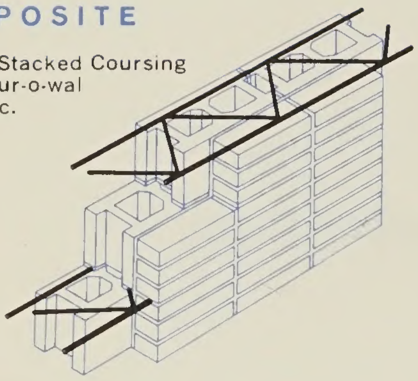
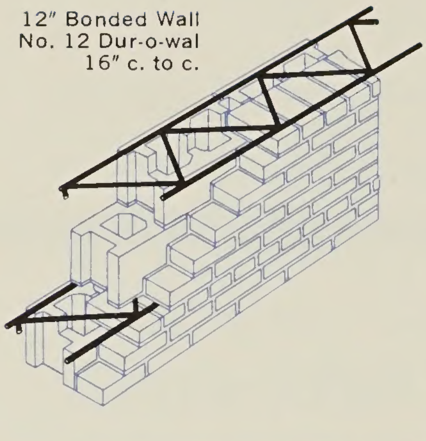
Basic Dur-O-waL Applications

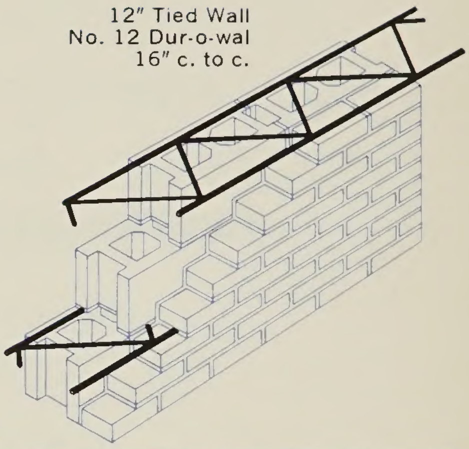
COMPOSITE



12″ Wall Stacked Coursing
No. 12 Dur-o-wal
16″ c. to c.

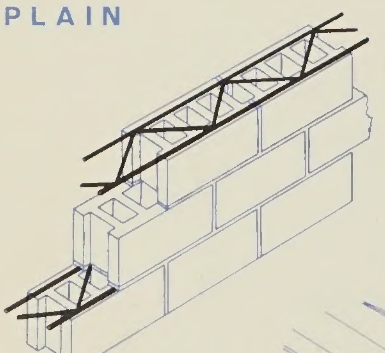


12″ Bonded Wall
No. 12 Dur-o-wal
16″ c. to c.

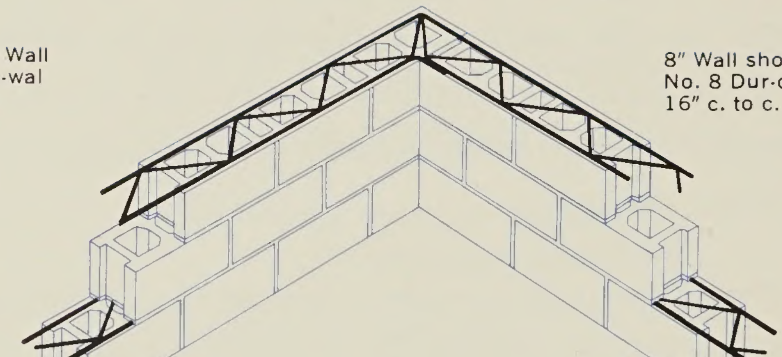


12″ Tied Wall
No. 12 Dur-o-wal
16″ c. to c.

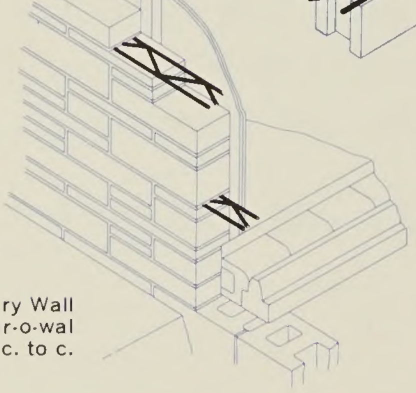
PLAIN



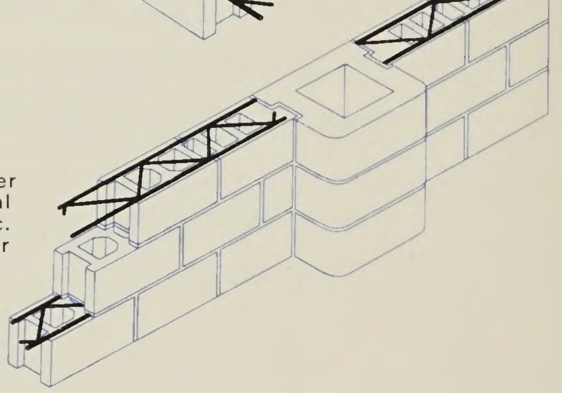
8″ Masonry Wall
No. 8 Dur-o-wal
16″ c. to c.



8″ Wall showing Corner
No. 8 Dur-o-wal
16″ c. to c.

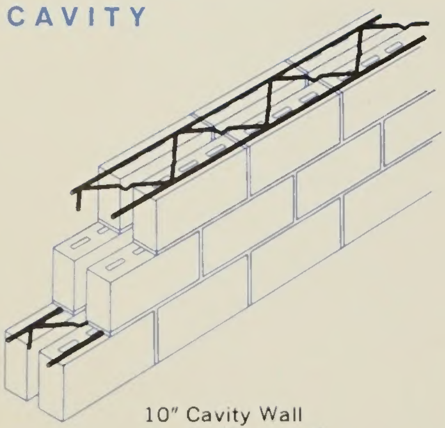


6″ Coursed Masonry Wall
No. 6 Dur-o-wal
16″ c. to c.

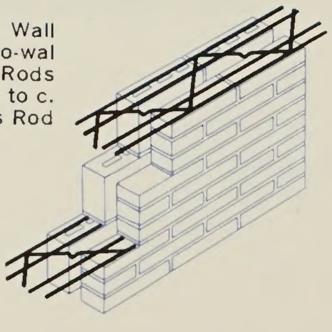


8″ Wall with Pilaster
No. 8 Dur-o-wal
16″ c. to c.
Control Joint at Pilaster

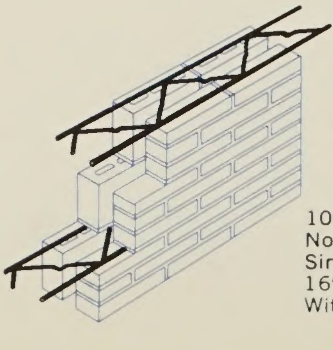
CAVITY



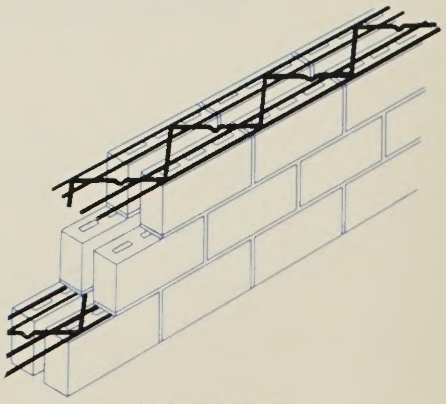
10″ Cavity Wall
No. 10 Dur-o-wal
Single Side Rods
16″ c. to c.
With Drip Cross Rod



10″ Cavity Wall
No. 10 Dur-o-wal
Double Side Rods
16″ c. to c.
With Drip Cross Rod



10″ Cavity Wall
No. 10 Dur-o-wal
Single Side Rods
16″ c. to c.
With Drip Cross Rod



10″ Cavity Wall
No. 10 Dur-o-wal
Double Side Rods
16″ c. to c.
With Drip Cross Rod

SECTION 1

Use in Combination with Control Joints

As a point of departure for this discussion we list the following typical questions asked by architects:

1. "Should vertical control joints be used?"

2. "Should plaster be applied directly to concrete block in stack bond, reinforcing 16″ on center?"

3. "What is the best treatment for hairline cracks in plaster that has been applied directly to concrete block?"

4. "How frequently would you recommend the use of vertical control in long block walls in excess of 12′ in height?"

These questions involve many variables, and it is almost impossible to give pat answers. However, basically, all the questions deal with the problem of cracking, to which all types of masonry units are subject, in varying degrees, from the following causes acting separately or in combination:

(a) Unequal settlement or distortion of footings or structures supporting the masonry.

(b) Overstressing of masonry caused by external forces of unexpected severity, which may occasionally develop from earth pressures, earthquakes, hurricanes, etc.

(c) Abnormal distortions, especially expansion of pertinent elements such as structural frames, floors, and roofs.

(d) Volume change in masonry involving principally temperature and moisture shrinkage.

Items (a), (b), and (c) can be controlled by careful design, such as providing for adequate footings and foundations, and related design details.

The problem involved in item (d) has been summed up very well by R. E. Copeland, Consulting Engineer (formerly Director of Engineering for the National Concrete Masonry Association), in the following two paragraphs from a paper which appeared in the Journal of the American Concrete Institute.

> *"The subject of shrinkage and temperature stresses in masonry walls involves many separate phases, variable factors and complexities. It does not lend itself to neat mathematical solution. We know that masonry that is trying to contract due to internal changes, but is fully or partially prevented from doing so, will be stressed in tension and usually also in shear. We know that cracks form when these stresses exceed the strength of the masonry. Beyond this point we emerge into the field of theoretical speculation illumined only slightly by experimental data and systematic observation.*
>
> *"The principal approaches to minimizing shrinkage and temperature cracking in concrete masonry include: (1) improving the dimensional stability of the concrete itself and of the masonry, (2) reducing residual shrinkage by the use of properly dried units, (3) controlling crack width by means of reinforcing, and (4) incorporating wall design details which allow the masonry to contract more freely and without being overstressed."*

Improvement in regard to dimensional stability and residual shrinkage is definitely the responsibility of the masonry industry. Great strides have been made within the last few years and a continuing program of research is under way in this direction.

Practical experience indicates that control of crack width and the wall flexibility that allows masonry to contract more freely can both be achieved by architect and engineer in the design of a masonry wall — with a judicious use of horizontal Dur-o-wal joint reinforcement in combination with vertical control joints.

Horizontal joint reinforcement — when, where, and how much? In combination with how many control joints?

The amount of horizontal joint reinforcement to be used depends largely upon the type of mortar. When Type N mortar is used, there is a limit to the amount of stress load that mortar will carry. Wall panels designed with Type N mortar can be reinforced effectively up to the point where Standard weight Dur-o-wal is used every second course, or 16″ o.c. The use of steel beyond this point would not appear to be justified. When using Type M or Type S mortar, however, the strength of the wall increases uniformly as the amount of steel increases, and Dur-o-wal can be effectively used in every course.

The effective spacing of control joints naturally also depends on type of mortar and amount of steel used. When Type N mortar is used, with Standard weight Dur-o-wal in every second course 16″ o.c., control joints should be spaced approximately 20 feet o.c. An increase of 25% in this spacing is reasonable, to allow the architect to work the control joints into his architectural treatment more readily.

When using *Type M or S* mortar, control joints should be spaced approximately 30 feet apart with reasonable allowance for architectural treatment requirements.

Within and in addition to the maximum space limitations mentioned in the preceding, control joints should be placed at points of stress concentration, such as:

1. Change in wall height.
2. Change in wall thickness.
3. At chases in walls required for pipes and columns.
4. Abutment of columns and walls.
5. Over weakened planes in foundation walls.

As far as the control joint itself is concerned, the architect is concerned with three principles:

1. The joint must cut the masonry wall completely from top to bottom, so as to form a truly stress-relieving joint.
2. It must be structurally sound in that a sufficient shear section is developed to provide for lateral stability.
3. It must be either self-sealing or one that can be very easily caulked to prevent water penetration or leakage.

Masonry manufacturers are now producing ready-made control joint units to meet all of these requirements. Another, meeting with growing nation-wide popularity is the *Rapid Control Joint*, developed by the producers of Dur-o-wal. This is a pre-molded rubber-type unit, engineered to meet the requirements with special emphasis on a complete self-sealing action.

Referring back to the four questions at the beginning of this analysis:

Should control joints be used? Yes. They are definitely helpful for the control of cracking in masonry walls, especially when used in combination with proper horizontal joint reinforcement.

Should plaster be applied directly to concrete block in stacked bond, reinforcing 16″ on center? If stacked bond walls are designed properly, there is no reason why they should not be plastered direct, the same as for running bond walls.

What is the best treatment for hairline cracks in plaster that has been directly applied to concrete block? In a survey of projects where this condition occurred, it has been noted that in all cases, sand gypsum plaster base was used. Where lightweight plaster such as Perlite or Vermiculite was used, this condition did not appear to exist. Therefore this type of plaster is recommended for direct-to-concrete use.

How frequently would you recommend the use of vertical control in long block walls in excess of 12′ in height? Walls of this height have a relatively greater resistance to cracking than average-size walls, and control joints can be spaced somewhat farther apart than normally recommended. In no case, however, should they be spaced more than 2 times the height. It is good practice to space control joints in high walls an average of 1.5 times the height.

The mortars are designated as Type M, Type S, and Type N respectively, in A.S.T.M. standard C-270-59T.

Type M—1 Masonry Cement, 1 Portland Cement, not less than 4½ Sand nor more than 6 Sand

or 1 Portland Cement, ¼ Hydrated Lime, not less than 3 Sand nor more than 3¾ Sand

Type S —1 Masonry Cement, ½ Portland Cement, not less than 3½ Sand nor more than 4½ Sand

or 1 Portland Cement, ½ Hydrated Lime, not less than 3½ Sand nor more than 4½ Sand

Type N—1 Masonry Cement, not less than 2¼ Sand nor more than 3 Sand

or 1 Portland Cement, 1 Hydrated Lime, not less than 4½ Sand nor more than 6 Sand

Specific recommendations for use of Dur-O-waL appear in Section 7, Page 17.

Specific recommendations for use of Control Joint appear in Section 6, Page 15.

Use in Composite Masonry Walls

(Dur-O-waL in lieu of headers)

SECTION 2

The following material summarizes the results of an "Investigation of Continuous Metal Ties as a Replacement for Brick Ties in Masonry Walls" conducted by the Armour Research Foundation for the makers of Dur-o-wal. The objective of this research was to provide factual data in support of the idea, commonly accepted by architects and engineers, that the conventional header course in composite masonry walls is a definite point of weakness. Another method of tying the two wythes of the wall together, without interrupting the continuity of the wall, and without a point of stress concentration, is certainly welcome to the building industry. Especially when this goes hand in hand with a reduction in flex-cracking, drier walls, less efflorescence, lower costs and greater design freedom. The complete details of this Armour Research Foundation study are available in a 44-page booklet which we will be glad to give you on request.

INTRODUCTION

Building codes usually require that no less than 4% of the area of a two-wythe masonry wall be composed of headers. While all recognized national codes provide for the use of metal ties in lieu of headers, some local code officials still restrict their use in load bearing walls because they feel that sufficient information has not been made available to show that the substitution of ties for headers will not weaken the wall.

The objective of this study was to provide information on the relative merits of continuous reinforcement vs. header courses in brick and concrete block wall construction. The phases studied were compressive strength, flexural strength in the vertical direction, and water permeability.

PROCEDURE

WALL CONSTRUCTION AND WORKMANSHIP. Specimens for this investigation were constructed by a practicing mason using first class techniques; all joints, including collar joints, were filled with mortar. The walls, four feet wide by eight feet high and of two thicknesses, eight and twelve inches, consisted of a brick face with either four or eight inch concrete block back-up. The wythes were tied together with either a standard header every seventh course or a continuous wire tie every sixth joint. Three walls were built for each condition to be studied; all were allowed to cure for at least 28 days before testing.

MATERIALS. Two metal ties were studied, a tab type consisting of two parallel wires with a four inch wire tab every 15 inches, and a truss type. Both reinforcements were fabricated from No. 9 gauge wire. At the collar joints these ties provide 0.021 and 0.0196 sq. in., respectively, of steel per square foot of wall area.

The back up consisted of expanded slag, autoclaved concrete units of the appropriate size. The blocks were sampled and tested for compressive strength in accordance with ASTM C140-52.

Table I — COMPRESSIVE STRENGTH OF AUTOCLAVED CONCRETE BLOCK

Size (inches)	Compressive Strength, psi (gross area)					Average psi
	1	2	3	4	5	
4 x 8 x 16	1010	930	880	940	940	940
8 x 8 x 16	950	1330	1100	1030	1100	1100

The face of the walls consisted of a hard-burned face brick; flexural and compressive strengths and initial rates of absorption (suction) were obtained in accordance with ASTM C67-50.

Table II — PROPERTIES OF FACE BRICK

Property Measured	Sample No.					Average
	1	2	3	4	5	
Compressive strength, psi	19,300	16,750	14,900	13,500	14,200	15,750
Flexural strength, psi	732	732	738	609	738	710
Suction, gm/minute	5.66	18.09	8.92	9.50	14.03	11.24

Table III — MORTAR PROPERTIES

Wall Group	Average Compressive Strength, psi	Flow Rate
Compressive strength	834	115
Transverse strength	978	108
Water permeability	864	115

These brick have compressive strengths which place them in the highest classification units.

All walls were laid with a type N mortar conforming to ASTM C270-59T. (Minimum average compressive strength of 750 psi for two-inch cubes made with mortar having an initial flow of 100 to 115.)

Table IV COMPRESSION DATA OF BRICK AND BLOCK WALLS

Wall Thickness (inches)	Ultimate Load, Lbs.			Init. Comp. Failure, psi			Ult. Comp. Failure, psi			Young's Elastic Modulus, psi x 10^{-6}		
	H[1]	S[2]	T[3]	H	S	T	H	S	T	H	S	T
8	238,500	262,000	345,000	390	390	390	622	681	898	1.99	2.60	2.82
12	315,000	347,000	375,000	318	361	361	543	602	650	1.34	1.12	0.88

1—Header brick tie 2—Special Dur-o-wal tie 3—Standard Dur-o-wal tie

COMPRESSION TESTS. Tests were made on walls of both thicknesses constructed with brick headers and both types of wire ties. Since three specimens were tested for each condition, a total of eighteen walls were evaluated. Eccentric line loading, applied one-third the wall thickness in from the block face, was employed.

TRANSVERSE TESTS. The number and types of walls tested in flexure were the same as those described in the section on compressive strength. Transverse strength in the vertical span of the walls was determined.

WATER PERMEABILITY STUDIES. Water penetration was observed on six eight-inch brick and block walls three of which were tied with header brick (HT) and three with the truss-type reinforcement (WT). Wind driven rain was simulated.

Two wind pressures, 20 and 35 psf, were used to study the rate of penetration. First experiments were made at the lower pressure; however, after eight days no leaks appeared and the pressure was increased to 35 psf for two additional days. This procedure was followed for both a wire-tied and a header-tied wall. The remaining specimens were studied under 35 psf pressure for seven days. The 20 psf (77 mph wind) and 35 psf (somewhat greater than 100 mph) pressures were chosen because they are commonly noted in design specifications for lateral force loadings.

CONCLUSIONS

COMPRESSIVE STRENGTH. Table IV lists the average results of the compression investigations. Analysis of the data indicates that the average standard deviation of the ultimate compressive strength of the wire-tied walls is 40 psi.

The data show that initial failure occurred at approximately the same stress for all of the eight-inch walls. Initial failure of the 12-inch walls occurred at a somewhat lower stress, however, the 12-inch walls will carry greater loads due to their greater cross-sectional areas. This phenomenon is in accord with the statistical theory of brittle failure which states that the larger the volume of the structure the greater the probability that a critical flaw will initiate failure. Ultimate stress values show similar relationships.

Crack patterns of the walls were typical of masonry compression failure. Initial failure is defined as the load at which either crushing or shear cracking first occurs. As the load was increased, vertical shear cracks appeared throughout the wall. Loading was continued until the balance arm of the testing machine indicated that the unit would support no additional load. With the approach of ultimate failure, crushing was pronounced in the block and was starting in the brick.

The ultimate strength data seem to indicate that wire ties are more effective than brick ties in holding a wall together after initial cracking. However, to be significant, the average strength of one group must exceed that of another by 177 psi. Statistical comparisons then show that the eight-inch walls reinforced with truss type ties are significantly stronger than the eight walls built with equivalent wall ties or with headers. The three groups of 12-inch walls show no significant differences, but inspection of the data reveals a similar trend.

TRANSVERSE STRENGTH. Statistical analysis of the data shows that there is no real difference in transverse strength in the vertical span associated with the type of wall tie. The strength differences shown in Table V are within the normal variation inherent in masonry construction. To be significant, there would have to be an average stress difference of at least 25.4 psi between the eight-inch wall groups and 8.4 psi between the 12-inch wall groups.

Table V VERTICAL SPAN TRANSVERSE LOAD DATA

Wall Thickness (inches)	Property	Type of Tie		
		Header	Special	Standard
8	average load, psf	150	157	143
	standard deviation, psf	26.5	23.1	11.5
	coefficient of variation, %	17.6	14.7	8.1
	average stress, psi	98.5	103.0	94.0
	modulus of elasticity, psi x 10^{-6}	2.26	1.34	1.30
12	average load, psf	198	171	177
	standard deviation, psf	2.9	16.2	18.8
	coefficient of variation, %	1.5	9.4	10.6
	average stress, psi	57.7	50.0	51.6
	modulus of elasticity, psi x 10^{-6}	1.23	1.66	1.97

Failure patterns indicated uniform loading with breaks occurring at or near a center course but independent of either wire tie or header course location.

WATER PERMEABILITY STUDIES. No water came through the block face wire-tied wall WT 1, until the air pressure was raised from 20 to 35 psf after eight days exposure. Leakage did not occur in the WT 2 wall until after five days exposure. Typical behavior for wire-tied walls was the appearance of moisture in the collar joint after about eight hours exposure. The wet areas in the collar joint would continue to spread until, after five or six days, wet areas would appear on the block face. Movement of the moisture appeared to be caused by slow absorption into the mortar.

For the header-tied walls (HT 1, 2, 3) moisture appeared on the back of the specimens within the first hour of exposure. Within two or three days the ends of the header brick showed considerable wet areas and moisture was beginning to appear in the block joints. It appeared that the brick-mortar interface acts as a duct to transfer moisture through the wall. Although water tended to collect in the collar joint the same as in the wire-tied walls, the headers acted as a bridge to carry the moisture across this barrier. All of the header-tied walls accumulated considerable water in the joints and cores of both the brick and the block. This was not true of the wire-tied walls where most of the moisture was confined to the collar joint.

CONCLUSIONS

The purpose of this investigation was to compare structural and moisture transmission properties of brick and block masonry walls using continuous wire ties opposed to header ties. The method of obtaining the data were adaptations of standard procedures used in studying masonry construction.

Conclusions to be drawn from the compression study are:

1. Initial failure load is not affected by the type of wall tie,
2. After the initial failure, wire ties are more effective in holding a wall together than header ties, and

The vertical transverse loading studies demonstrated that continuous horizontal wire reinforcement is as effective as brick headers in tieing walls against lateral loading. This is in contrast with compression findings where the wire reinforcement takes up some of the shear stress which is characteristic of compression failure in masonry. The explanation is that vertical transverse strength is a function of mortar-masonry bond. This bond is not affected by the horizontal reinforcement.

Measurements and observations of water permeability of the masonry specimens indicate a marked difference in their permeabilities. Wire-tied walls do not provide a continuous path of water to travel through the wall. The collar joint, a major barrier, is capable of soaking up considerable quantities of water before the moisture is transmitted into the block mortar joints. In contrast, the header brick-mortar interface provides a continuous path through the wall, and under wind pressure, seepage may occur in an hour or two as compared to several days for a wire-tied wall.

These results demonstrate that continuous wire reinforcement can replace header bricks without detriment to the structural capability of the wall. In fact, it provides advantages such as easier construction and the attainment of less permeable walls.

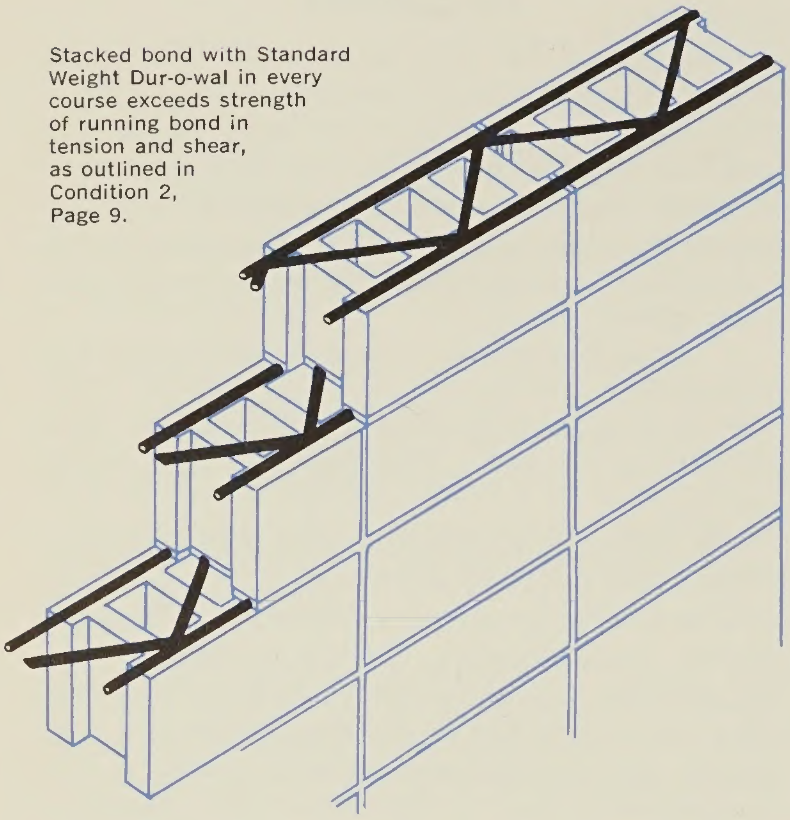
Specific recommendations for use of Dur-O-waL appear in Section 7, Page 17.

Specific recommendations for use of Control Joint appear in Section 6, Page 15.

SECTION

3

Use in Stacked Bond Masonry



Stacked bond with Standard Weight Dur-o-wal in every course exceeds strength of running bond in tension and shear, as outlined in Condition 2, Page 9.

With the use of Dur-o-wal joint reinforcement, stacked bond masonry walls can be designed with load-bearing properties that equal or exceed running bond, and meet all requirements of the codes.

To arrive at the additional tensile strength which brings this about and provides distribution of load stresses over the entire wall, we have taken into account the following structural factors:

1 Compressive strength under vertical loads, either axial or eccentric, is similar for stacked bond and running bond construction.

2 In stacked bond masonry, concentrated loads are carried down to the support by a particular tier of masonry without distribution to adjacent masonry. This need not jeopardize wall stability, if allowable working stresses are not exceeded. But heavy concentrated loads should be supported on pilasters or the masonry otherwise strengthened. This is a problem of good design that can be solved with Dur-o-wal.

3 When a transverse load is applied uniformly and is transmitted in the vertical span to horizontal reactions there is no measurable difference between stacked bond and running bond construction in transverse load resistance. This is a function of the horizontal mortar joints and is not influenced by whether the vertical joints are staggered or continuous.

4 The horizontal distribution of transverse load stresses to vertically extending reactions is nil in the case of unreinforced stacked bond masonry. Bending resistance in the horizontal span must be provided by embedded joint reinforcement or other means. However, it should be noted that the proportion of wind and other transverse loads transmitted to vertically extending reactions usually is small compared to that transmitted to top and bottom edges of the wall. This is due to the openings and wider spacing between vertical reactions and supports as compared to horizontal reactions and supports. Stacked bond masonry, therefore, is not seriously inferior to running bond in transverse strength, and can be at least equally strong if reinforced.

5 In regard to faced walls, facing units laid in running bond provide some horizontal stiffness of the stacked bond back-up if the facing and back-up are properly bonded, but nevertheless the back-up should be reinforced.

6 To insure over-all stability, load-bearing walls and exterior walls of stacked bond masonry should be reinforced horizontally, so their ultimate tensile strength in this direction equals the strength of ordinary running bond construction. Some horizontal reinforcement is desirable even in non-load-bearing stacked bond partitions.

To determine the horizontal reinforcement needed in stacked bond masonry to equal running bond, two comparative conditions can be considered:

1 The horizontal reinforcement needed in stacked bond masonry as compared with running bond without reinforcement.

2 The horizontal reinforcement needed in stacked bond masonry as compared with running bond with normal joint reinforcement in every second course, 16″ c. to c.

The basis for answering these conditions can be found in the table, reproduced on page 10 in this book, from the Uniform Building Code, 1958 Edition. This is the only recognized code that includes working stresses for flexural tension and shear in unreinforced masonry. The working stresses are shown in pounds per square inch of gross or net area, as indicated, for compression, tension in flexure, and shear, with respect to various types of mortars, masonry units and construction.

GENERAL EFFECTIVENESS OF DUR-O-WAL FOR STACKED BOND HOLLOW BLOCK CONSTRUCTION

Using these Uniform Building Code working stresses and 20,000 psi working stress for high tensile steel, it was found that stacked bond masonry with Standard Weight Dur-o-wal in every second block course (16″ c. to c.) develops measurably more strength in both tension and shear than unreinforced running bond, with Types M, S, or N mortars*

Stacked bond masonry with Standard Weight Dur-o-wal in every block course develops measurably more strength in both tension and shear than running bond reinforced every second course, with Types M, S, or N mortars.

A masonry wall 10 ft. high (15 block courses) was the basis for comparison in arriving at these conclusions. The shear value for running bond in a wall of this height is 3300 lb. total. Shear value for Dur-o-wal in every second course is 5180 lb. total. This combination equals 8480 lb. total shear value.

The shear value for a stacked bond wall of this height, using Types M, S or N mortars, with Dur-o-wal in every course (8″ c. to c.) is 10,360 lb. total, a greater value than the preceding.

In view of this analysis it is reasonable to make the following recommendations:

SPECIFIC DUR-O-WAL RECOMMENDATIONS FOR STACKED BOND CONSTRUCTION

For condition 1—to strengthen stacked bond equal to unreinforced running bond masonry: Basically, use Standard Weight Dur-o-wal Masonry Wall Reinforcement every second block course, 16″ c. to c. However, an additional band of reinforcing is recommended near the top of the wall, the critical area for load distribution. Therefore it is recommended that Standard Weight Dur-o-wal be used in top three courses, 8″ c. to c., and in every course 16″ c. to c. throughout remainder of the wall.

For condition 2—to strengthen stacked bond equal to running bond masonry reinforced every second course: Standard Weight Dur-o-wal used every course, 8″ c. to c., is more than adequate.

---

*Original ASTM C-270, and mortar type designations A, A-1 or B were in effect prior to 1954. Since 1954 these designations are M, S or N, respectively.

*Allowable Working Stresses in Unreinforced Unit Masonry***

MATERIAL Grade of Unit	**TYPE A MORTAR*** **			**TYPE B and C MORTAR*** **		
	Compression	**Tension in Flexure or Shear**		**Compression**	**Tension in Flexure or Shear**	
Special Inspection Required	No	Yes	No	No	Yes	No
Solid Brick Masonry						
4500 plus psi	250	20	10	200	15	7.5
2500 to 4500 psi	175	20	10	140	15	7.5
1500 to 2500 psi	125	20	10	100	15	7.5
Solid Concrete Unit Masonry						
Grade A	175	12	6	140	12	6
Grade B	125	12	6	100	12	6
Grouted Masonry						
4500 plus psi	350	25	12.5			
2500 to 4500 psi	275	25	12.5			
1500 to 2500 psi	225	25	12.5			
Hollow Unit Masonry	85	12*	6*	70	10*	5*
Cavity Wall Masonry Solid Units						
Grade A or 2500 psi plus	140*	12*	6*	110*	10*	5*
Grade B or 1500 to 2500 psi	100*	12*	6*	80*	10*	5*
Hollow Units	70*	12*	6*	50*	10*	5*
Stone Masonry						
Cast Stone	400	8	4	320	8	4
Natural Stone	140	8	4	100	8	4
Gypsum Masonry	20	—	—	20	—	—
Unburned Clay Masonry	30	8	4	—	—	—

*Net area. **Allowable working stresses psi gross cross-sectional area (except as noted).

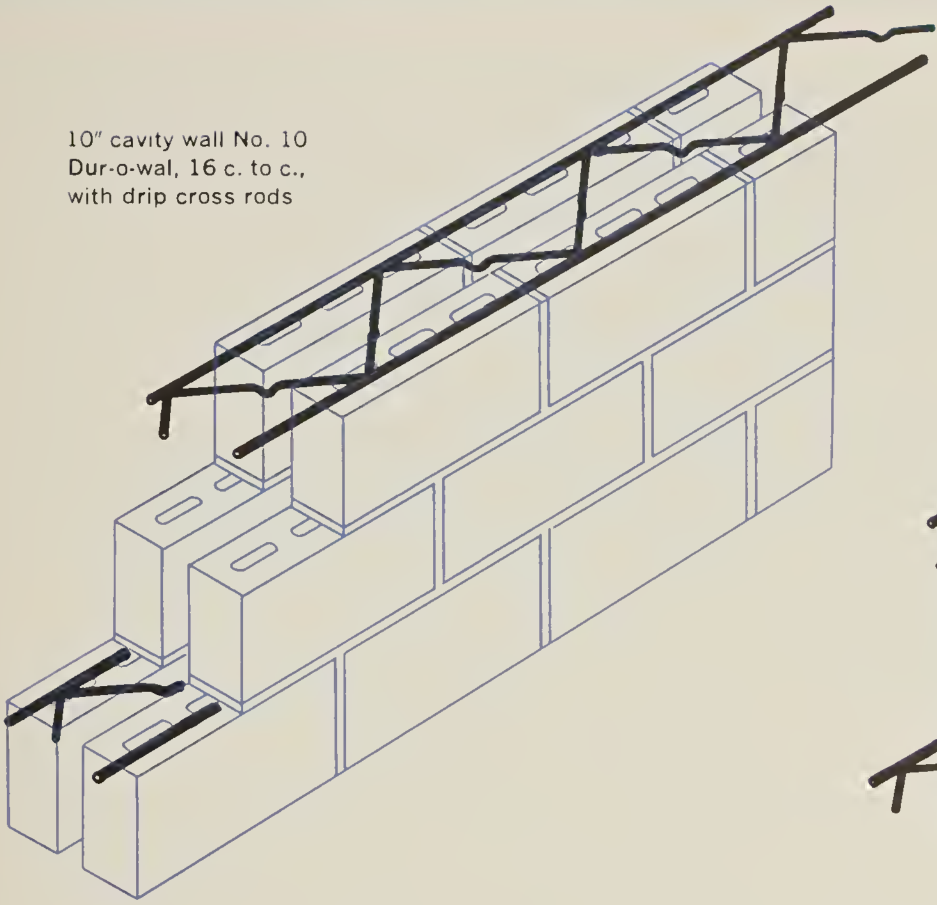
***Required minimum compressive strength of mortars: Type A, 2000 p.s.i.; Types B and C, 750 p.s.i.

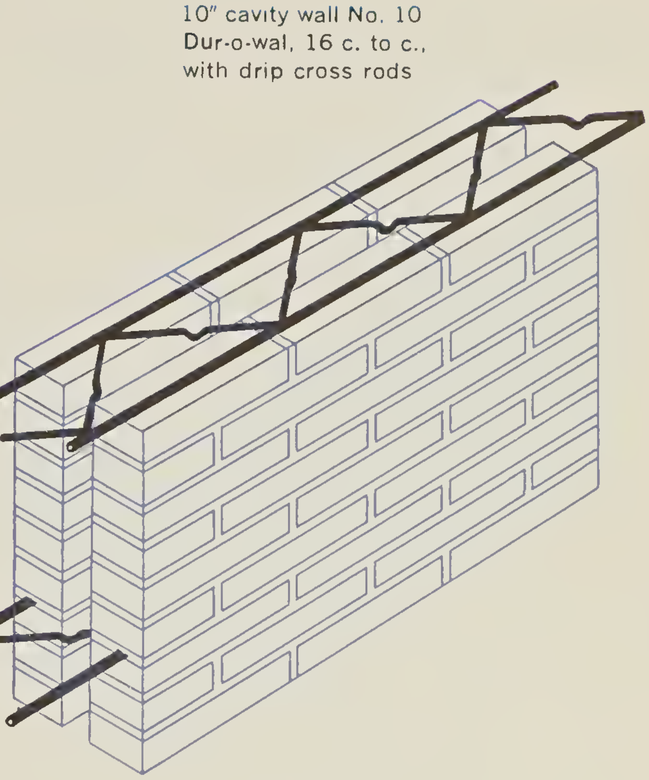
The above is copied from the Uniform Building Code, 1961 Edition.

SECTION

4

Use in Cavity Walls





This analysis started with a careful search of generally accepted codes to determine the proper amount of high tensile steel ties needed to insure structural stability in cavity walls.

FROM THE CODES

In the 1961 edition of the Uniform Building Code of the International Conference of Building Officials, the section regarding cavity wall masonry includes the following:

> *"The facing and backing of cavity walls shall be bonded with three-sixteenths-inch (3/16″) diameter steel rods or metal ties of equivalent stiffness embedded in the horizontal joints. There shall be one metal tie for not more than each four and onc-half square feet (4½ sq. ft.) of wall area. Ties in alternate courses shall be staggered and the maximum vertical distance between ties shall not exceed eighteen inches (18″), and the maximum horizontal distance shall not exceed thirty-six inches (36″). Rods bent to rectangular shape shall be used with hollow masonry units laid with the cells vertical; in other walls the ends of ties shall be bent to 90-degree angles to provide hooks not less than two inches (2″) long. Additional bonding ties shall be provided at all openings, spaced not more than three feet (3′) apart around the perimeter and within twelve inches (12″) of the opening. Ties shall be of corrosion-resistant metal, or shall be coated with a corrosion-resistant metal or other approved protective coating."*

This is identical, except for wording, with other national and regional codes such as the American Standards Code, etc. An exception is the Southern Standard Building Code which specifies a more conservative spacing of ties — one for each three square feet of wall surface.

Consequently some local codes call for 3/16-inch Z ties spaced one for each three square feet, or the double 3/16″ diameter rectangular, or U-type tie.

The more typical as well as the more conservative code requirements are the basis for our evaluation of the use of Dur-o-wal in cavity wall construction.

CROSS-SECTIONAL AREA OF STEEL AND STRESS DISTRIBUTION

In the comparison table on page 13, listing various types of ties spaced according to the conservative codes, it is shown that Dur-o-wal spaced 16″ c. to c. vertically, provides more cross-sectional area of high-tensile steel than any of the other ties listed and in addition, reinforces the masonry longitudinally.

Also, because of Dur-o-wal's trussed design, the stresses that affect cavity wall construction are evenly distributed to both wythes. The two wythes are firmly tied together and tend to act more nearly as a unit. This is not true with Z ties and only partially true with rectangular type ties.

CORPS OF ENGINEERS

In the Omaha field laboratory, the Corps of Engineers conducted a series of Tests of the flexural strength of concrete block walls in horizontal bending. The test program included six panels 8 ft. long and 3′ 4″ high, of 4-2-4 cavity wall design. Three panels were tied with ¼″ Z-bars, one for each 3 sq. ft. of wall area. The remaining three panels were tied with Extra Heavy* Dur-o-wal in each bed joint, 8″ c. to c.

Panels were broken in a structural steel jig which reproduced conditions of a simple span with a concentrated load in the center. The load was applied with a hydraulic jack through a vertical breaking beam. The breaking beam was suspended from a roller chassis which was part of the jig. Test panels were built on rollers made of short sections of pipe, eliminating an estimated 90% of the normal bottom restraint.

Quoting the report of engineers who recorded the results, here is what happened:

Z-BARS VERSUS DUR-O-WAL REINFORCEMENT

The test results were reported by F. W. Cox and J. L. Ennenga, Corps of Engineers, in a paper published in the Journal of the American Concrete Institute, May, 1958. The three cavity walls built with Z-ties cracked and failed at loads producing bending moments of 840, 1040 and 720 ft.-lb. per ft. of wall height. The three cavity walls reinforced and tied with Dur-o-wal cracked at a load producing an average bending moment of 1200 ft.-lb. per ft. of wall height and after cracking continued to sustain additional increments of load, their ultimate bending resistance exceeding 1600 ft.-lb. per ft. of wall height or approximately double the bending resistance of the unreinforced Z-tied walls.

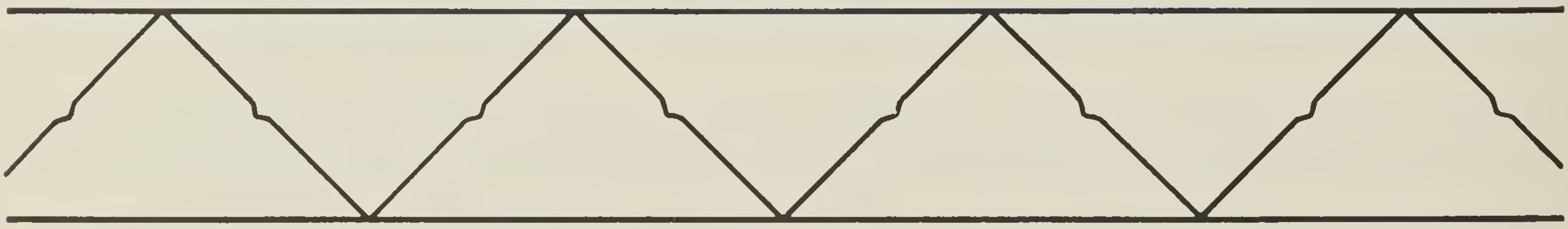
Regarding resistance to flexural cracking the authors explained the superiority of the Dur-o-wal reinforced walls as follows: "The joint reinforcement unites the two wythes and tends to cause them to act as a single beam." They observed that the Z-bar tied walls failed, ". . . in two stages, beginning with rupture of the loaded wythes." Also, that, "Initial failure occurred at an average moment of 860 ft.-lb. per ft. of height. The far wythe continued to carry an average moment of 480 ft.-lb. for a perceptible time, indicating that the two wythes act as separate beams."

CONCLUSIONS

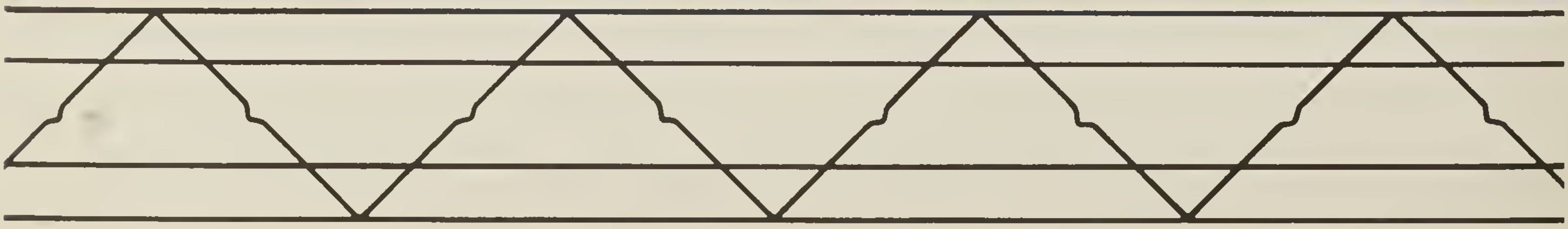
The Corps of Engineers tests show that cavity walls reinforced with Extra Heavy Dur-o-wal in every course outperform an unusually conservative Z-bar tied design by almost two to one in ultimate bending resistance. This indicates that typical code requirements for the tying of cavity walls can be met with Standard Dur-o-wal in every second course or 16″ o.c. since Standard and Extra Heavy Dur-o-wal are identical as to the size and spacing of the trussed diagonals which serve as ties. Also, in the case of Dur-o-wal, tie anchorage is positive and anchorage stresses are well distributed through the side rods over the entire mortar joint area. Such anchorage obviously is not obtained with Z-bars and certain other types of ties.

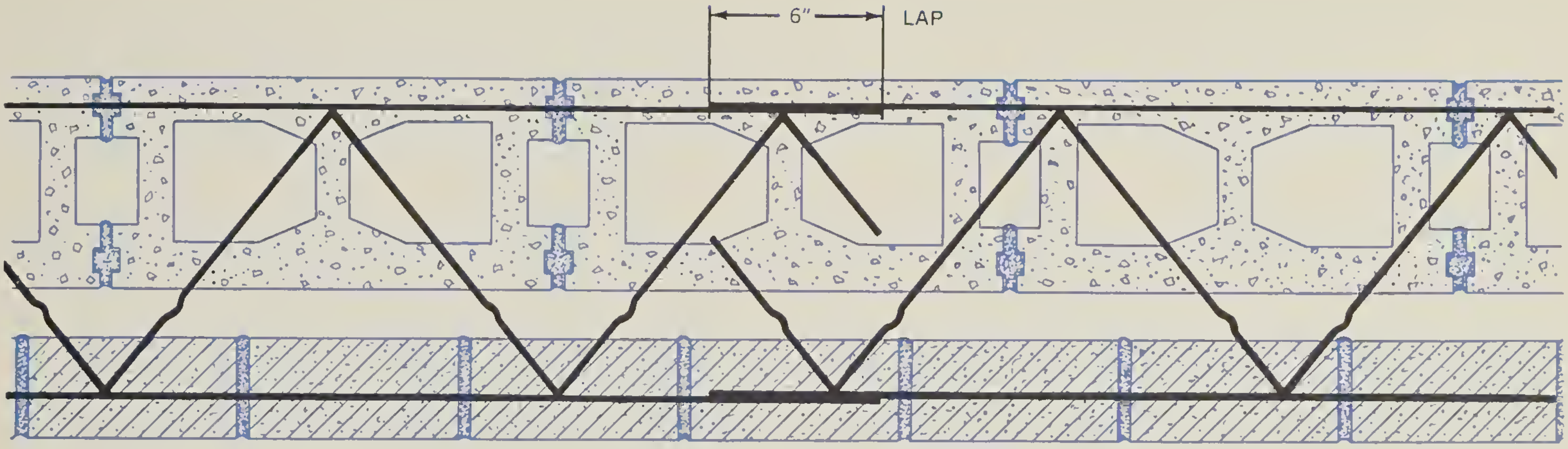
*3/16″ side rods, 9 gauge cross rods.

Dur-O-waL cavity wall tie – single side rods



Dur-O-waL cavity wall tie – double side rods



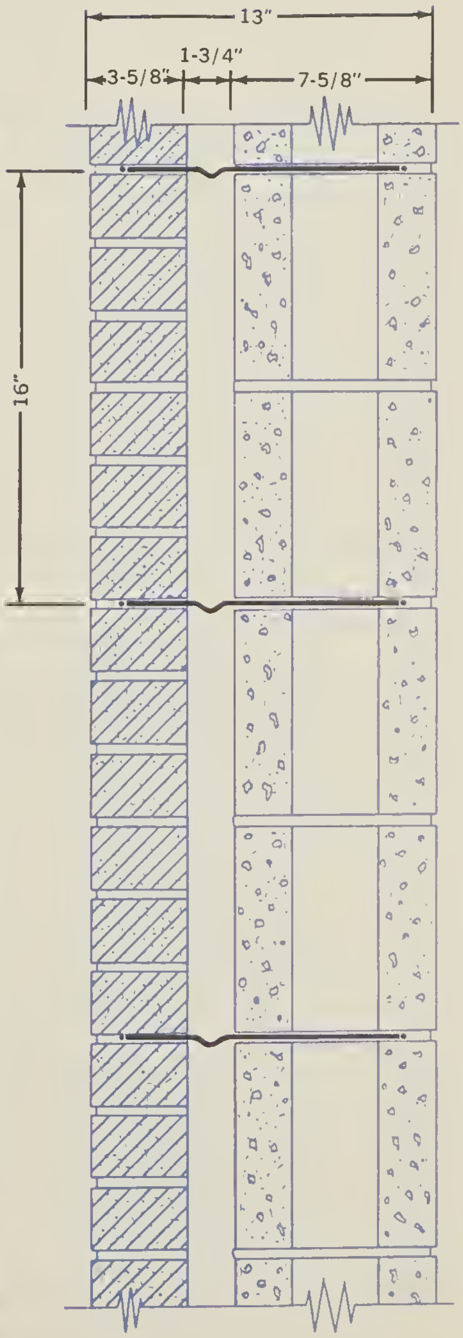


Codes generally call for a minimum of one 3/16″ dia. tie for each 3 to 4½ sq. ft. of wall. Dur-o-wal exceeds this requirement and provides maximum anchorage with positive and uniform spacing of ties. It also reinforces the masonry longitudinally.

Comparison Table—Various Type Cavity Wall Ties

Type	Spacing	Cross Sectional Area of Ties per sq. ft. Wall
Z-Ties, 3⁄16″ dia.	1 for each 3 sq. ft. 1 for each 4½ sq. ft.	.0092 sq. in. .0061 sq. in.
Rectangular ties, two 3⁄16″ dia.	1 for each 3 sq. ft. 1 for each 4½ sq. ft.	.0184 sq. in. .0122 sq. in.
U-Ties, two 3⁄16″ dia.	1 for each 3 sq. ft. 1 for each 4½ sq. ft.	.0184 sq. in. .0122 sq. in.
Dur-o-wal, No. 9 gauge	**Spaced 16 in. o.c. vertically**	**.0196 sq. in.**

Dur-o-wal for Cavity Walls is furnished with steel side rods and galvanized cross rod or with all galvanized steel rods.



SECTION
5

Use with Glass Block

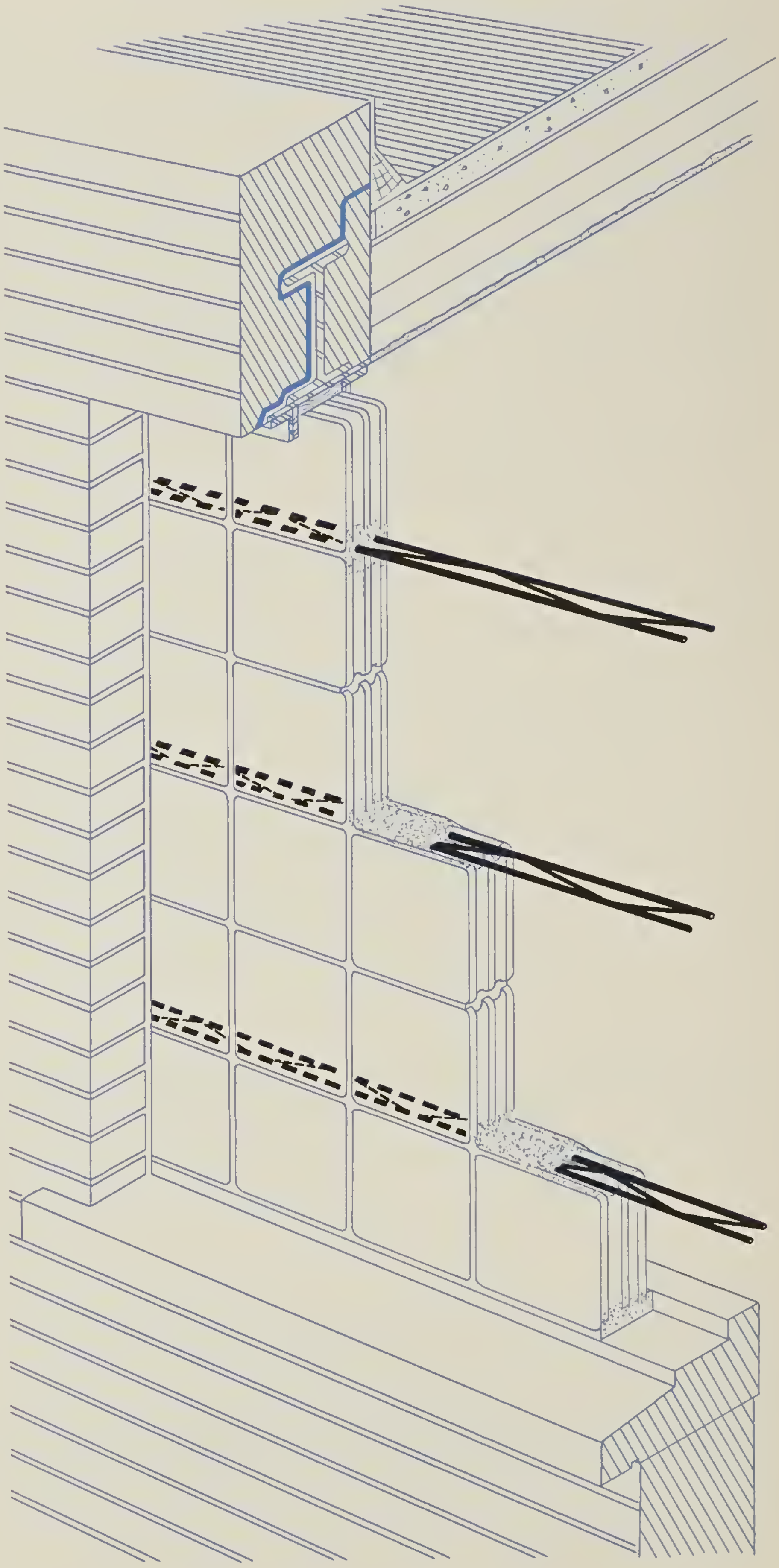
RECOMMENDATION AND DESCRIPTION

Standard Weight Dur-o-wal No. 3-S is recommended for glass block reinforcement. It is manufactured with a No. 9 gauge trussed designed Cross Rod butt welded to No. 9 gauge deformed Side Rods. Double mortar locks at each weld add greatly to the tensile load carrying capacity of the assembly. All rods are in a single plane thus assuring a neat, tight mortar joint.

Dur-o-wal for glass block reinforcement is manufactured from galvanized steel. It weighs 177 lbs. per 1,000 feet.

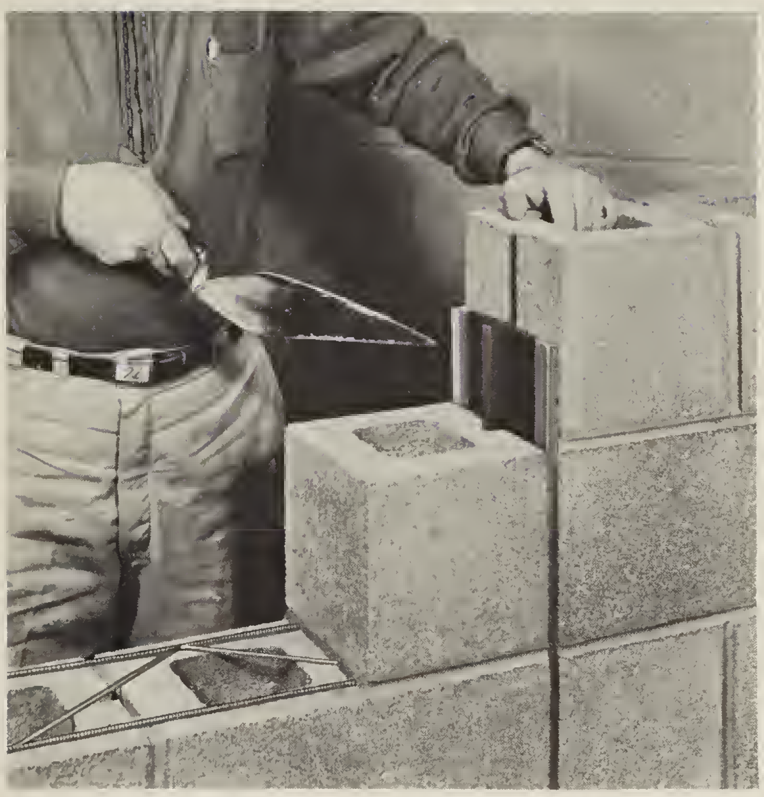
SUGGESTED GENERAL SPECIFICATION

Glass block panel reinforcement shall be Dur-o-wal (or approved equal) with deformed side rods and trussed designed cross rod and shall comply with material requirements of ASTM Standard A-82-61T for cold drawn steel wire for concrete reinforcement. Reinforcement shall extend from end to end of mortar joints but not across expansion joints, with any unavoidable joints spliced by lapping the reinforcement not less than 6 inches. Dur-o-wal shall be spaced not more than 2 feet apart vertically. In addition reinforcement shall be placed in the joint immediately below and above any openings within a panel. The reinforcement assembly shall consist of two parallel longitudinal galvanized, deformed steel wires, No. 9 gauge, spaced approximately 2 inches apart, and having butt welded thereto No. 9 gauge trussed designed cross wire at intervals not exceeding 8 inches, or the equivalent approved by the building official.



Wide Flange and Regular Rapid Control Joint

SECTION
6



The Rapid Control Joint comes in sections, four block courses in depth. Quickly, easily installed, it saves time and money.

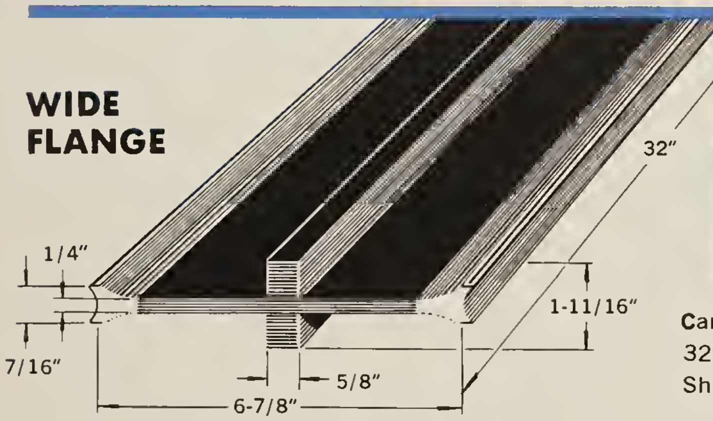
QUICKEST, MOST EFFECTIVE MEANS TO A YEAR-ROUND WEATHER SEAL

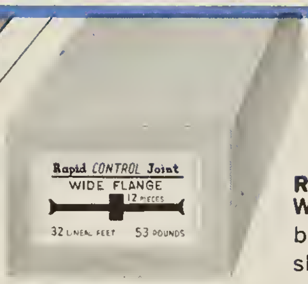
The wide flange Rapid Control Joint has been engineered by the makers of Dur-o-wal to provide a complete answer to the control joint problem. Flexible neoprene flanges, that take care of the full working thickness of a concrete block, expand and contract with the joint — to make-up a quickly installed control structure that lets a wall "breathe," yet keeps itself sealed tight. Caulking is not always necessary in standard concrete block construction. The neoprene compound used is resistant to flex-cracking, weather (ozone), heat and cold. Not harmed by oil or solvents. The concave edge-allows easy compression.

The regular (narrow flange) Rapid Control Joint is recommended by construction engineers for various special needs.

Both types of Rapid Control Joint fit neatly and quickly into the grooves of metal sash jamb block.

WIDE FLANGE



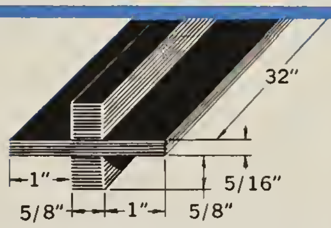


Carton Contents: 12 pcs. 32" long, total 32'. Shipping weight 53 lbs.

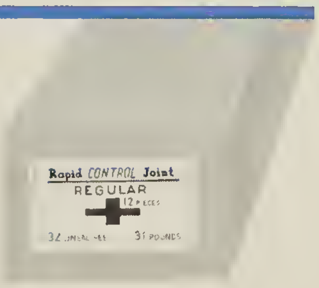
GENERAL SPECIFICATIONS

Regular
Wide Flange Rapid Control Joint when used in combination with Dur-o-wal Masonry Wall Reinforcement, shall be spaced approximately 20 feet c. to c. for average height walls, and not to exceed 2 times wall height for high walls. Incorporated within limitations of spacing noted above, control joints should also be placed at points of great stress concentration such as—change in wall heights; change in wall thickness; at chases in wall required for pipes, vents and columns; abutment of column and walls, and over weakened planes in foundation walls.

REGULAR



Carton Contents: 12 pcs. 32" long, total 32'. Shipping weight 31 lbs.



PRODUCT SPECIFICATIONS

Rubber material in Regular and Wide Flange Rapid Control Joint complies with applicable ASTM and Federal Specifications.

Regular

75-85 Shore Durometer hardness for Shear Section.

Wide Flange

75-85 Shore Durometer hardness for Shear Section.

25-35 Shore Durometer hardness for Neoprene compound Flange.

Shear Strength Investigation of Wide Flange and Regular Rapid Control Joint Material

The following gives you the results of an investigation conducted by the Armour Research Foundation in regard to the shear strength of two masonry control joint materials and of the lateral strengths of masonry joints in which these materials are used. These materials consisted of a regular moulded rubber joint, and a similar joint with a soft neoprene flange vulcanized to each leg.

Shear strength was determined in an assembly consisting of three eight-inch slotted concrete cubes arranged so that the control joint material could be placed in the adjacent slots of the two joints between the three blocks. The blocks were held together to prevent spreading during load application.

Shear strengths of the joint materials amounted to 541 and 470 psi for the regular joint and the wide-flange joint respectively.

I. INTRODUCTION

Control joints are built into masonry walls to provide relief for wall movements caused by temperature and moisture changes. The control joints under study are of two types, 1) Rapid Control Joint Regular and 2) Rapid Control Joint Wide Flange. These materials are products of Dur-o-wal. The investigation consisted of determining the resistance of these materials to lateral forces.

II. EXPERIMENTAL PROCEDURE

A. Control Joints. The regular control joint consists of a moulded stiff rubber cross shaped section. A ⅝ x ⅝ inch leg fits into the slots in notched metal window sash block. A $\frac{5}{16}$ x 1 inch leg fits into the joint between adjacent blocks.

The wide flange control joint consists of a moulded stiff rubber center section, similar to the regular joint with a neoprene flange vulcanized to the joint leg. The purpose of the neoprene flange is to seal the joint without the use of a caulking material.

B. Test Assemblies. Dense concrete blocks, 8x8x8 inches, were cast with ⅝ x ⅝ inch slots on their opposite sides. Three of these units were placed together with the control joint material fitting into the adjacent slots of the two joints thus formed. This assembly was used as the basic test unit for these experiments.

For direct shear measurements, the three units were held snugly together with long pipe clamps to prevent spreading of the blocks during the test. The end units were blocked up, clamped, and the center unit loaded. The assembly was orientated so that the load was perpendicular to the long direction of the joint, thus introducing a shear force through the rubber control joint. Load was applied until a maximum was indicated by the testing machine indicator dial. Three assemblages were investigated for each material.

III. RESULTS AND DISCUSSION

Table I lists the results of the shear tests on the two control joint materials. Failure occurred when the rubber material sheared in one or both of the slots. The results indicate that the regular control joint has about 15% greater shear strength than the wide flange control joint.

Table I

SHEAR STRENGTH OF CONTROL JOINT MATERIALS

Control Joint Type	Load per 8-inch of Joint, lbs.			Average Load lbs.	Shear Strength psi
	1	2	3		
Wide Flange	2250	2540	2260	2350	470
Regular	2220	2850	3050	2706	541

IV. CONCLUSIONS

If a wall 20 feet between column supports resists a 20 psf wind load the shear force at the column is

$$V = (20)/2(20) = 200 \text{ lbs./ft. of wall}$$

The shear stress in psi of such a loading for ⅝ inch thick shear key is

$$S = \frac{(200)}{(12)(.625)} = 26.5 \text{ psi}$$

This value is considerably below the shear strengths of the joint material studied. If no other factors beside direct shear is involved these moulded sections will make excellent control joint material.

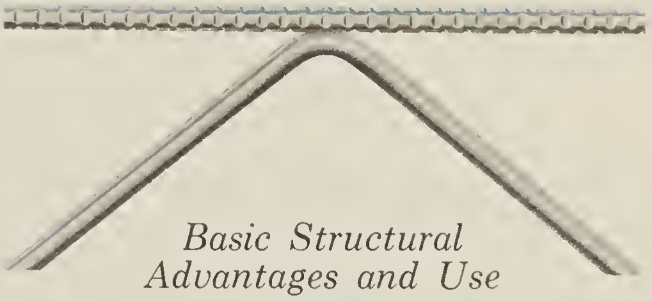
SECTION
7

Dur-O-waL Truss Design and Butt-welded Construction

Here are the basic general specifications for the use of Dur-O-waL

Wall reinforcement shall be Dur-o-wal Truss Design (or approved equal) with deformed side rods and shall comply with material requirements of ASTM Standard A-82-61T for cold drawn steel wire for concrete reinforcement. Minimum weight of joint reinforcement shall be 0.178 pounds per lineal foot. When cast in regular mortar joints, deformation of side rods shall be of such that side rods develop a minimum surface bond stress of 700 p.s.i. with Type M and 400 p.s.i. with Type N mortars.

Dur-o-wal shall be placed in first and second bed joints (8″ c. to c.) above and below openings and in every second bed joint (16″ c. to c.) throughout remainder of structure. Dur-o-wal in first bed joint immediately above and below openings shall be continuous. In second bed joint it shall extend two feet beyond each side of opening. Reinforcement shall be lapped sufficiently at splices (6″ minimum) to insure continuity and corners shall be either prefabricated or cut and bent as shown in details. Dur-o-wal shall not pass through vertical masonry control joints, except where required for structural reasons as noted on plans.



Basic Structural Advantages and Use

Dur-o-wal is a steel reinforcing member designed specifically for use as reinforcement in masonry walls. It is manufactured from high-tensile steel, deformed and electrically butt-welded on modern automatic machinery to insure uniform quality. Low-cost, proved performance, nationwide distribution have resulted in Dur-o-wal being specified for all types of masonry wall construction.

List of Mechanical Features

- **Mechanical bond every 8 inches of wall**
- **Two mortar locks at each weld**
- **Deformed side rods lay straight and flat**
- **Electric butt welds place all rods in a single plane**
- **Trussed design causes side rods to work together**
- **More pounds of working steel for greater wall strength**
- **10 foot lengths for easy handling**
- **6 inch laps at splices develop continuity and maintain reinforcing strength**
- **Deformed side rods for maximum bond strength**
- **All steel in single plane makes possible uniform mortar joints**
- **Conforms to ASTM Standard A-82-61T for high tensile steel**

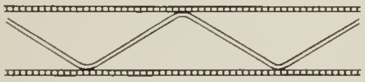
Dur-O-waL Types and Sizes

Dur-o-wal is packaged for easy handling in ten-foot lengths, 50 pieces per bundle, with protective end wraps.

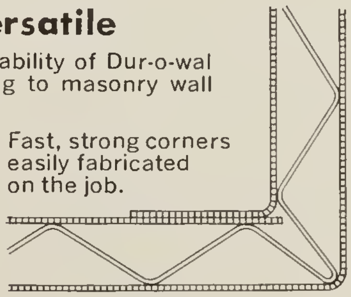
Dur-o-wal is available in two basic weights, **Extra Heavy** and **Standard.** Extra Heavy Dur-o-wal is manufactured with 3/16 inch side rods and No. 9 gauge cross rods. Standard Dur-o-wal is manufactured with No. 9 gauge side rods and No. 9 gauge cross rods. Both are manufactured to cover a wide range of different wall thicknesses.

Dur-O-waL is Versatile

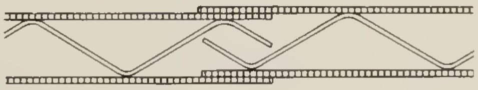
These drawings illustrate the adaptability of Dur-o-wal to all structural problems pertaining to masonry wall reinforcement.



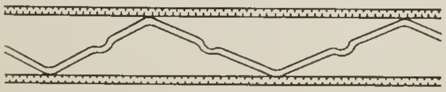
Exclusive Truss Design for maximum lateral strength.



Fast, strong corners easily fabricated on the job.



Full 6 inch lap at the splices maintains reinforcing continuity.



Drip-Section Dur-o-wal for cavity wall construction.

Prefabricated corners and tees (all sizes) also available.

Selection Table

Wall Thickness	4 in.	6 in.	8 in.	10 in.	12 in.	13 in.
Extra Heavy Dur-o-wal...	4-EH	6-EH	8-EH	10-EH	12-EH	13-EH
Wt. per 1000 lin. ft. (lbs.).	247	250	257	266	276	282
Standard Dur-o-wal...	4-S	6-S	8-S	10-S	12-S	13-S
Wt. per 1000 lin. ft. (lbs.).	178	180	187	196	207	212

Furnished in either bright basic or galvanized steel. Specify Drip Section Dur-o-wal for Cavity Walls.

NOTE—In determining whether to specify Standard weight or Extra Heavy weight Dur-o-wal, comparisons in following table should be properly evaluated:

	Pounds tensile steel per lin. ft.	Surface bond area—sq. in. per lin. ft.	Number of mortar locks per lin. ft.
Standard weight Dur-o-wal....	0.19	11.18	3
Extra Heavy Dur-o-wal........	0.26	14.14	3

SECTION

8

Dur-O-waL Deformed Side Rods

Dur-o-wal's deformed side rod magnified to show the similarity of its deformations to those of Hi-Bond reinforcing steel bars.

To function with optimum effectiveness, the steel reinforcing member for masonry wall construction must not merely be strong in itself, but must also provide for the best possible gripping contact with the mortar in which it is embedded. Research has clearly indicated the need for improved deformation of side rods. The makers of Dur-o-wal consider the development of such improvement important, and have given the subject special attention.

Specifications for Deformation of High-tensile Steel Wire

With specially designed machinery, the depth and spacing of deformations on four surfaces of the wire are controlled to a high degree of accuracy.

Dur-O-waL specifications are as follows:

Side rods shall be deformed, and shall conform in general with Minimum Requirements for the Deformation of Deformed Steel Bars ASTM A 305-50 T, more specifically described as follows:

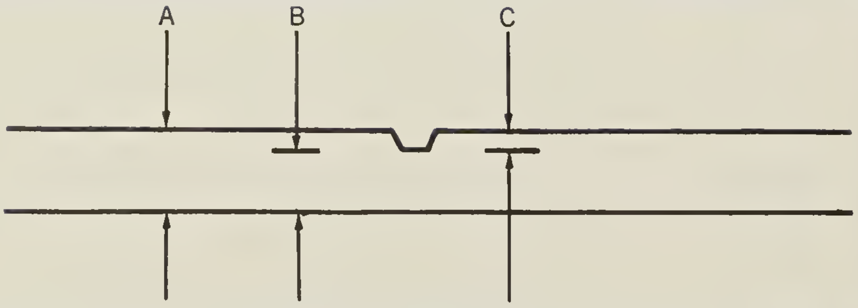
A—Deformations shall be spaced along the bar at substantially uniform distances. The deformations on opposite sides of the bar shall be similar in size and shape.

B—The perimeter of the bar shall be marked with four distinct deformations. The overall length of deformations shall be such that the summation of gaps between ends of the deformations shall not exceed 25% of the nominal perimeter.

C—The average spacing between each set of deformations shall be approximately .7 of the nominal diameter of the bar, or not less than eight sets per inch; thus forming 32 separate and distinct deformations per inch.

D—The height, and/or depth, of deformations shall be .024 inches for 3/16" rods, .022 for #8 gauge and .018 inches for #9 gauge rods.

Micrometer Measurement of Deformation



A—Overall Measurement After Deformation

B—Bottom of Deformation to Outside Surface

C—Required Depth of Deformation

Size of Wire	Diameter of Wire Inches	A	B	C
3/16	.1875	.192	.168	.024
#8	.1620	.165	.143	.022
#9	.1483	.150	.132	.018

Direct measurement of -B- equal to Table above will give required depth of deformation -C-.

Effect of Deformation on Strength of Rods

Report of Tests—
Plain and **Deformed** Wire

At—Toledo University Research Center
By—Prof. E. Saxer
Date—Aug. 2, 1956
Data—Comparative samples taken from same coils

Wire specification—
Standard for Cold Drawn Wire ASTM A 82
Minimum Tensile Strength **80,000** psi

Specimen	Type	Average Diam. Inches	Average Load lbs.	PSI Stress*	Net Sectional Area Sq. In.
3/16	Plain	.1883	2540	91,212	.027848
3/16	Def.	A-.1920	2446	87,834	.023388
		B-.1727			
		C-**.0193**			
9 Gauge	Plain	.1477	1490	86,963	.017134
9 Gauge	Def.	A-.1503	1378	80,426	.011215
		B-.1281			
		C-**.0222**			
8 Gauge	Plain	.1623	1894	91,549	.020688
8 Gauge	Def.	A-.1643	1802	87,101	.014042
		B-.1414			
		C-**.0229**			

*Based on sectional area of plain wire.

Test Results

Specimen	Cross-Section Area Sq. In.		Total Load Lbs.	Tensile Stress PSI based on	
	Gross	Net		Gross Section	Net Section
3/16 Plain	.027848	.027848	2540	91,212	91,212
3/16 Def.	.027848	.023388	2446	87,834	104,583
9 Gauge Plain	.017134	.017134	1490	86,963	86,963
9 Gauge Def.	.017134	.011215	1378	80,426	122,871
8 Gauge Plain	.020688	.020688	1894	91,549	91,549
8 Gauge Def.	.020688	.014042	1802	87,101	128,329

Note—Increase in Tensile Stress in reduced section indicates effect of Cold Working process of Rolling in Deformations.

SECTION
9

Quality Comparison Guide

All claims made by the manufacturers of Dur-o-wal rest squarely upon the proved superiority of this brand of masonry wall reinforcement, in comparison with other types. **There are three points of importance in comparing quality: 1—*Weight of material*—actual weight per 1000 lineal feet—flexural strength in relation to weight of steel in wall. 2—*Deformation*—effect on surface bond—effect on wire strength. 3—*Mortar locks.***

1. *Weight of Material*

a. Research tests clearly indicate that when Class A mortars are used, the flexural strength of walls increases uniformly as the weight of steel increases.

See Report of Flexural Tests on Following Pages.

b. Comparative weights — No. 8 for 8″ Wall.

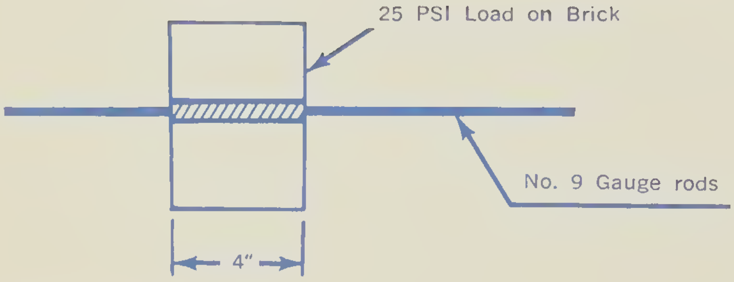
Identification	Wt/1000′	Size Side Rods	Size Cross Rods
Truss Type No. 8 Extra Heavy	257	3/16″	#9 Gauge
Other Type No. 8 Heavy	196	3/16″	#9 Gauge
Truss Type No. 8 Std.	187	#9 Gauge,	#9 Gauge
Other Type No. 8 Std.	140	#9 Gauge	#9 Gauge

NOTE: Truss type extra heavy Dur-o-wal is 31% heavier than other extra heavy types. Truss type standard Dur-o-wal is 33% heavier than other standard types.

2. *Deformation*

a. Comparative tests

No. 9 Gauge Rods	4″ Concrete Brick
4″ Embedment	25 PSI Load on Brick
⅜″ Mortar Joints	



***CLASS A MORTAR**

Dur-o-wal (Deformed)
"No slip" Load 1455 lbs. Av. — Rods Failed on Tension.
Bond Stress — Exceeds 780 lbs. sq. in. surface area.
Maximum bond stress — not determined as rods failed.

Plain Rods Load 905 lbs. Av. — Rods pulled out.
"slip" Maximum bond stress — 375 lbs. sq. in. surface area.

Other Type (Deformed)
"slip" Load 1285 lbs. Av. — Rods pulled out.
Maximum bond stress — 680 lbs. sq. in. surface area.

Conclusions—
Dur-o-wal (deformed rods) had 108% more bonding value than plain rods.
Dur-o-wal deformation was more than 14.7% more effective than other type.

***CLASS B MORTAR**

Dur-o-wal (Deformed)
"slip" Load 825 lbs. Av.—Rods pulled out.
Maximum bond stress—435 lbs. sq. in. surface area.

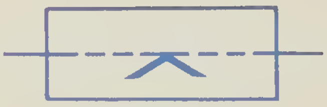
Plain Rods
"slip" Load 430 lbs. Av.—Rods pulled out.
Maximum bond stress—230 lbs. sq. in. surface area.

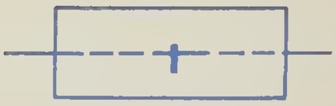
Other Type (Deformed)
"slip" Load 465 lbs. Av.—Rods pulled out.
Maximum bond stress—245 lbs. sq. in. surface area.

Conclusions—
Dur-o-wal (deformed rods) had 89% more bonding value than plain rods.
Other type (deformed rods) had only 7% more bonding value than plain rods.
Dur-o-wal deformation was 77% more effective than other type.

3. *Mortar Locks*

Comparative test results—involving joint only
No. 9 Gauge Rods
Cast in ⅜″ mortar joints
4″ x 8″ Concrete Brick
Cross rods cut off ¾″ from side rods
Side rods neutralized with heavy coating of grease


Dur-O-waL Joint
Plan View


Other type Joint
Plan View

***CLASS A MORTAR**

"Load at Initial Slip"

Dur-o-wal	1145 lbs. Average
Other type	760 lbs. Average

Dur-o-wal 46% Greater

Same test repeated on Dur-o-wal with 3/16″ side rods and No. 9 cross rods
"Load at Initial slip" 1280 lbs. Average

***CLASS B MORTAR**

"Load at Initial Slip"

Dur-o-wal	665 lbs. Average
Other type	525 lbs. Average

Dur-o-wal 26% Greater

NOTE: "no slip" is defined as no movement, on needle of .0001 inch recording dial gauge, occurring at any time during test.

"Load at Initial Slip" is tensile force on rod at instant unloaded and registered movement of .0010 inches.

As a large percentage of masonry is laid up with *Class B mortar, the high values developed with Dur-o-wal double mortar locks and deformation of side rods is extremely important.

Full strength of side rods is developed by lapping 4″ in *Class A mortar and 8″ in Class B mortar. If lapped as per Dur-o-wal recommended detail 6″ lap for Class B is sufficient, because of locks in lap area.

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*Original ASTM C-270, and mortar type designations A, A-1 or B were in effect prior to 1954. Since 1954 these designations are M, S or N, respectively.

SECTION

10

Research Study *(University of Toledo)*

In an effort to obtain pertinent information as to how joint reinforcing actually affects the strength of masonry construction, Dur-o-wal sponsored a program of research carried on by the Research Foundation of the University of Toledo during the summer and fall of 1956.

Objectives of Study

The basic purpose of this project was to obtain as much information as possible about the behavior of horizontal joint reinforcing in concrete masonry construction.

More specifically, the program was designed to provide data on the following items:

1. The relative lateral strength of walls constructed with various types and amounts of reinforcing.

2. The relative effectiveness of various types and amounts of reinforcing when used in walls laid up with mortars of widely varying strengths.

3. The effect of deformations in the side rods on the bond with both weak and strong mortars.

4. The effect on bond of the joints formed by the side rods and the cross rods.

5. The effect of deformation on the strength of the side rods.

Scope of Study

Fulfillment of the objectives listed necessitated a program which was in several respects the most comprehensive ever attempted along this line. In some cases the work was conducted in accordance with routine methods, but in at least two instances the tests were made in a manner never before used to our knowledge. (Air bag for wall tests, and pullout tests on wires embedded in masonry joints.)

The most important data was that concerned with the effectiveness of reinforcing in actual construction. For this phase of the project, a total of thirty-nine walls, 9′-4″ by 4′-0″, were built and tested.

More than two dozen tension tests were made on plain or deformed wires of various sizes and types.

Almost 80 "pullout" specimens were tested in an effort to obtain an insight into the bond characteristics of various types and sizes of reinforcing steel.

Numerous tests were made to determine the pertinent properties of the block, sand, cement, and lime used in the construction of the walls. Batches of the mortar used for laying up the walls and the pullout specimens were tested in accordance with ASTM C-270.*

FLEXURAL STRENGTH OF WALLS IN HORIZONTAL SPAN

A summary of the results of flexural tests on walls is shown on the accompanying graph, on the next page, consisting of curves based on average strengths. The reference for strength is taken as the average load carried by four non-reinforced walls laid up in Class B mortar. All other values are expressed as a percentage of this reference value. Against the lateral strength are plotted the number of pounds of steel in the test panel. Test panels were six courses high and seven blocks long (9′-4″ x 4′-0″). The table identifies the various types of reinforcing.

The first point to be noted on the chart is the relatively small increase in lateral strength due to the use of strong mortar in non-reinforced walls, as indicated on the graph at points "a" and "b." These two points each represent the average of four test panels. The Class "A" walls* yielded an average strength about 15% greater than the specifications laid up with Class B mortar.* Both mortars were stronger than called for by ASTM standards, Class A showing compressive strength of 3540 p.s.i., and Class B 1100 p.s.i. While this difference in strength is only 15%, this is close to what should be expected. One must remember that the only difference in the walls lies in the mortar itself, and that the total thickness of the 5 layers of mortar represents only two inches of the total wall height of 48 inches.

When you examine the Class A curve, you will note that a vertical straight line is formed; all of the points shown on this line are for walls reinforced with Dur-o-wal, truss-type material. These points each represent the average of tests of at least two walls. There is an indication that the strength of these panels increases in direct proportion to the weight of steel in the wall. Note that the walls with five courses of reinforcing were 2.35 times as strong as the non-reinforced walls.

The curve for Class B walls indicates that there is not full utilization of reinforcing when a weaker mortar is used. The strength of the walls is partially determined by some factor other than the characteristics of either the block or the reinforcing as indicated by the curtailed difference in lateral strength. Careful examination of the walls after failure, revealed no failure in bond between the mortar and the steel. All evidence pointed to failure of bond between the Class B mortar and the block itself.

It would naturally follow that when using Class B mortar there is a certain amount of steel that would be of great benefit, but beyond this point greater quantities of steel would yield no comparable increase in strength. The graph indicates that maximum gain in masonry wall strength using Class B mortar is reached when reinforcing is used in every other course. The significance of this gain is apparent when noting that the truss-reinforced Class B walls are about 50% stronger than non-reinforced Class A walls, and 65% stronger than non-reinforced Class B walls.

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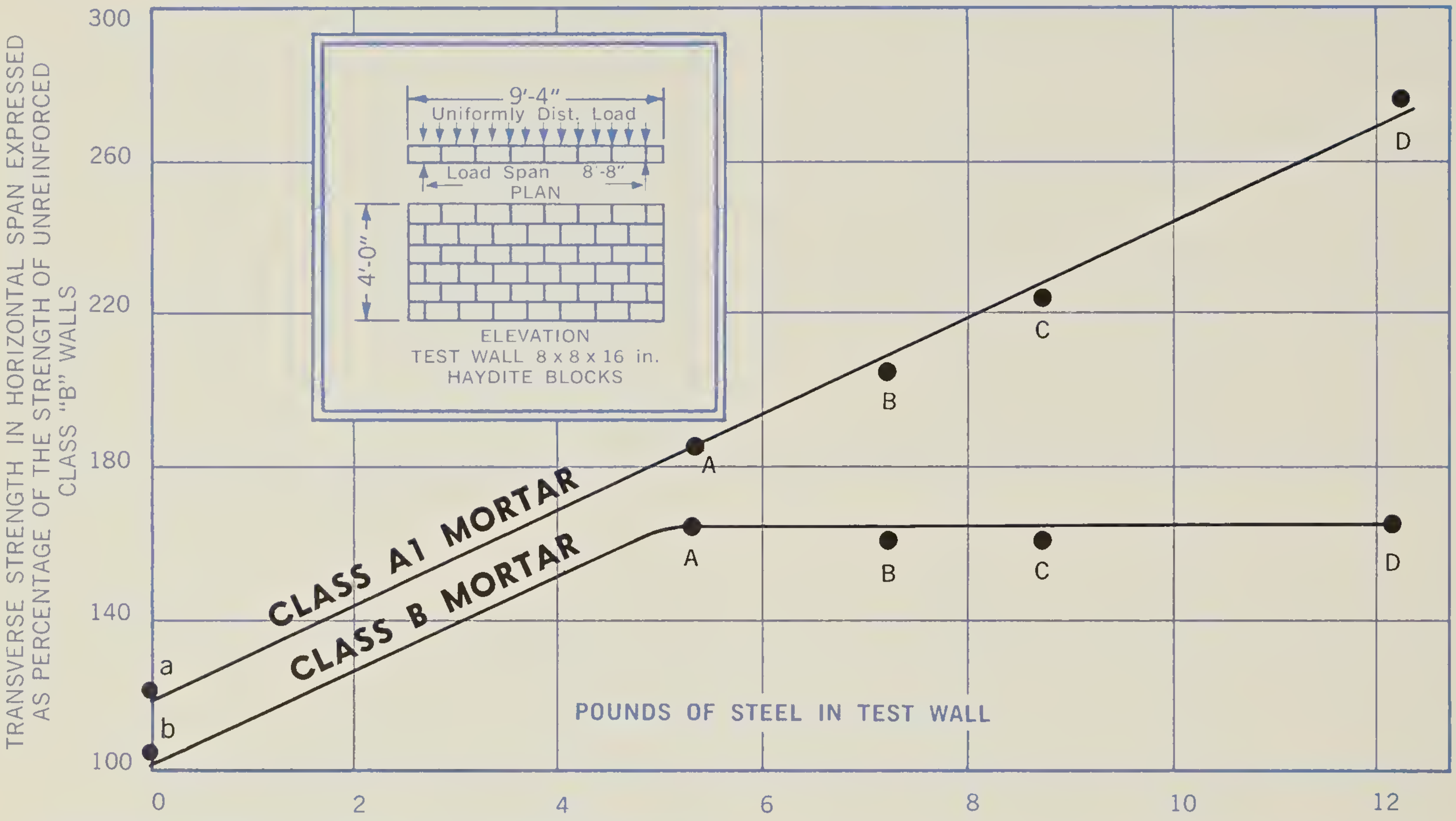
*The mortars are designated as Class A and Class B, respectively, in A.S.T.M. standard C-270-52T.

Proportions Class A1
By volume: 1 part cement—¼ part lime—3 parts sand
By weight: 55—6½—210

Proportions Class B
By volume: 1 part cement—1 part lime—6 parts sand
By weight: 55—26—420

Steel in Wall vs. Lateral Strength—Walls Reinforced With Dur-O-waL



	TRUSS RODS GAGE	SIDE RODS GAGE	TYPE REINFORCING	NO. OF JOINTS REINFORCED
A	9	9	T	3
B	9	3/16	T	3
C	9	9	T	5
D	9	3/16	T	5

Statement by Edwin L. Saxer

Professor and Chairman, Engineering Department
University of Toledo, November 5, 1956

1. The Dur-o-wal type of deformations are very effective in strengthening the bond between the mortar and the steel. The plain wires are capable of holding only about one-third as much load as the deformed rods.

2. The joint between the side rods and the cross truss forms an effective mechanical lock in the mortar although it is not strong enough to prevent slippage all by itself.

3. To a considerable extent the bond strength between steel and mortar depends upon the strength of the mortar. It appears that the bond in class A mortar is at least twice as great as that for class B mortar.

4. While the Dur-o-wal deformation process causes a reduction in cross sectional area the benefits of cold working are almost great enough to cancel out the effect of the reduced area. Even though the areas have been reduced by almost a third, there has been a reduction of only 5% in total strength.

5. In order to derive the maximum benefits from reinforcing, a high strength mortar should be used. Unless the bond between the mortar and the blocks is adequate, it is impossible to develop the full strength of the steel.

6. Where strong mortars are used, the strength of the wall increases as the amount of steel increases.

7. The distribution of tension stresses, and hence the distribution of cracks, is largely dependent upon the amount of reinforcing in the wall, although it is affected to some degree by the strength of the mortar.

8. There was little or no evidence to indicate that the presence of reinforcing weakened the horizontal joints. In fact, in several cases where there was a substantial horizontal joint crack, the failure occurred in an unreinforced joint.

SECTION

11

Galvanizing

Throughout the years since Dur-o-wal introduced a prefabricated reinforcing member for masonry walls, we have been asked frequently about the merits of galvanizing. The principal questions raised are discussed below.

EFFECT OF GALVANIZING ON BOND

The efficacy of portland cement concrete and of mortar in preventing the rusting of embedded steel is due in part to their alkaline constituents, principally calcium hydroxide. It is well known however that zinc is attacked by solutions of caustic alkalies such as calcium hydroxide. The following is taken from an Information Sheet, "Non-Ferrous Metals in Contact with Concrete," published by the Portland Cement Association.

> "When zinc is embedded in concrete, the moisture together with the calcium hydroxide react to form calcium zincate. The corrosion product forms a compact film around the metal and very firmly bonds it in the concrete. To what extent the zinc will be corroded cannot be stated with certainty. In those instances that have been observed, the reaction merely involved the outer layer of zinc and then ceased. It appears that the calcium zincate forms a dense, closely adherent film that protects the underlying metal from further attack. . . . In contact with dry, seasoned concrete, zinc will not corrode."

For the present, and until contrary evidence is developed, we are inclined to interpret the above statement as meaning that galvanizing will not seriously impair the bond of joint reinforcement with mortar and will continue to provide supplemental protection against corrosion of the steel.

EFFECT OF TIE DIAMETER ON CORROSION RESISTANCE OF GALVANIZED CAVITY WALL TIES

Modern code requirements for metal ties for cavity and multi-wythe walls contain an "equivalent stiffness" provision which permits approving No. 9 gauge ties provided they are properly spaced to impart the same degree of stiffness per square foot of wall area as 3/16" diameter ties used at the specified maximum spacing. This takes care of the structural aspects but does not quite answer the concern of some engineers regarding the comparative resistance to corrosion of No. 9 gauge and 3/16" diameter ties.

We have no evidence, and have seen none, that corrosion is a significant factor in the service life of galvanized metal ties regardless of whether they are No. 9 gauge or larger.

The question of the effect of wire size is discussed in the statements, reproduced below, from two metallurgists with long experience in the field of wire products. They conclude that for a given weight of zinc coating (fraction of an ounce per square foot of surface area) the corrosion resistance of galvanized No. 9 wire will be at least equal to that of galvanzed 3/16" diameter wire. Their comments on the capacity of zinc coatings to bleed over and protect welds will be of interest.

STATEMENT FROM NORTHWESTERN STEEL AND WIRE COMPANY

It was indeed a pleasure to discuss our product with you over the telephone yesterday. I only wish that we could have been in the same room, and much more time available to go into the many technical phases involved in the production of our products.

At your request the following detail is submitted.

I have had the inspection records made available on our galvanized wire production of material supplied to you. The two (2) sizes of wire in question were: (1) 9 ga. (.148" dia.) and (2) 3/16" (.187" dia. or 6¼ ga.). In order to clarify the weights-of-coating standards the type 1 portion involved of the table 7-20 page 46 of the A.I.S.I. Steel Products Manual for Carbon Steel Wire and Rods latest issue dated May 1957 is submitted.

Gage No. 5 and Coarser	Decimal Equivalent	Type 1 Coating Ounces of Zinc per sq. ft. of Surface Min.
#5 and coarser	0.207	0.65
6	0.192	0.50
7	0.177	0.40
8	0.162	0.40
9	0.148	0.40
10	0.135	0.30

It further states that fractional gages other than shown are produced with zinc coatings equivalent to those of next smaller size.

Our records indicate that the average weight of coating for 9 ga. wire is 0.47 oz./sq. ft., and 3/16" is 0.52 oz./sq. ft. So to begin with both wires are standard and in specification.

Resistance to corrosion is a function of the amount of zinc deposited upon the surface. By dividing the surface by the weight one comes up with a factor

$$9\text{ ga.}\ \frac{0.148 \times \pi}{\text{wt.}} = \frac{.465}{.40\ (\text{min. wt.})} = 1.16$$

$$\tfrac{3}{16}''\ \frac{0.187 \times \pi}{\text{wt.}} = \frac{.587}{.40} = 1.46$$

It becomes obvious that the smaller the ratio of the division to the surface area the larger will be the factor. For this reason then any factor number less than the two standard factor numbers, for the two sizes of wire will be an advantage over the standard, or over any combination of the two sizes. Now, to take the two specific cases.

$$9\text{ ga.}\ \frac{0.148 \times \pi}{\text{wt.}} = \frac{.465}{.47} = 0.99$$

$$\tfrac{3}{16}''\ \frac{0.187 \times \pi}{\text{wt.}} = \frac{.587}{.52} = 1.13$$

Quantitatively speaking the 9 ga. is 14% better to corrosion resistance in these two cases. To take the A.I.S.I. standards as written for the two sizes and based on the minimum values (i.e., 0.40 oz./sq. ft. minimum): the 9 ga. is 26% better than the 3/16" wire.

It can readily be seen that the protection on the 9 ga. is better than the 3/16" wire, from the protection standpoint 9 ga. may be used to advantage.

We also discussed the life of the cross wire welded product when performed on galvanized wire in regard to the resistance to corrosion at the weld. This we have been doing ourselves for 14 years. In such a plant as ours we have a number of ways to evaluate such a question.

(1) Observation during storage

(2) Observation in the field

(3) Observation of the mesh, some of which we use ourselves

(4) Customer contacts

During these 14 years we have never seen, or heard of premature corrosion at the welds. Zinc does possess the property of bleeding, or healing over, and therefore does not suffer the oxidation that one might suspect on first observing the product. It is my opinion that the life of the product will not be affected by the welding.

We are anxious to service our customers, please feel free to call upon us at anytime.

L. R. Dillenbeck, Supt. of Metallurgy

STATEMENT FROM THE COLORADO FUEL AND IRON CORPORATION

Mr. Harold Eyer, our District Sales Representative, has requested that I write you to discuss the corrosion resistance of 9 ga. versus $^{3}/_{16}$ inch galvanized wire. He further requested an opinion on the possibility of corrosion at the junction point of the longitudinal and transverse wires where they are welded.

The corrosion resistance of a galvanized piece of steel is a function of the amount of zinc (thickness of deposited layer) per unit of area. The American Iron & Steel Institute Steel Products Manual "Wire and Rods, Carbon Steel" lists these weights for various gages in Table 7-20 of their 1957 issue. Where the same gages occur similar weights are found in ASTM Specifications, Class 1 coating. These are given below:

7 ga.	.177 in.	.40 oz. Zn/sq. ft.
9 ga.	.148 in.	.40 oz. Zn/sq. ft.

Fractional gages, such as $^{3}/_{16}$″ (.1875″), are produced with the zinc coatings equivalent to the next smaller size. Thus, a $^{3}/_{16}$″ wire would be galvanized to a coating weight of a 7 ga. wire.

Since the thickness of the deposited zinc layer is the determining factor as regards the amount of zinc on the wire, one procedure would be to determine the volume relationship of the deposited zinc to that of the wire as shown by the formula

$$\frac{\pi dl \times .40/144}{\pi d^2 l/4} = \frac{.40}{36d}$$

for 9 ga. wire—

$$\frac{.40}{36 \times .148} = 0.0751 \text{ oz./cu. in.}$$

for $^{3}/_{16}$″ diameter wire—

$$\frac{.40}{36 \times .187} = 0.0594 \text{ oz./cu. in.}$$

Thus—

$\frac{0.0751}{0.0594} = 1.264$ or 9 ga. wire would have 26% more zinc per unit volume than $^{3}/_{16}$″ diameter wire.

Now for corrosion at the weld. When two pieces of galvanized wire are welded, the temperature of the weld is such that some or all of the zinc at the contact point is volatilized, as the weld temperature is well above the vaporizing temperature of zinc (1663° F.). This is evident as some of the zinc oxidizes and is present around the weld as a white powder.

Yet, I cannot recall a single instance where corrosion at the weld has been noticed or has been a source of trouble. We have made many tons of galvanized welded fabric which has been in both indoor and outdoor storage, and erected or fabricated where exposed to atmosphere and industrial fumes, but to my knowledge, corrosion at the welds has never been observed nor brought to our attention.

The absence of noticeable corrosion at these points could be explained by the cathodic protection offered by the zinc. It is known that a discontinuity in the zinc coating of a galvanized article will not necessarily result in corrosion at that point. The zinc adjacent to a break in the surface will sacrifice itself because of its electrochemical potential in relation to iron, thus protecting the base metal even though exposed. It is quite possible that such action takes place at the welds.

We are pleased to be of service to you and should further information be desired, please call upon us.

J. H. Shank, Supt. Quality Control Department

STATEMENT FROM NORTHWESTERN STEEL AND WIRE COMPANY

In answer to your question: If 9 gage has 0.40 oz. of zinc per sq. ft. of surface area, how could it possibly have more corrosion resistance than $^{3}/_{16}$ with 0.50 oz. per sq. ft. of surface minimum?

$^{3}/_{16}$ does not have a minimum requirement of 0.50 oz. per sq. ft. The 0.50 oz. minimum does not become effective until a full 6 gage wire size is produced. The minimum requirement stays 0.40 oz. on $^{3}/_{16}$ which is the same as the 9 gage minimum. This is spelled out at the bottom of table 7-20 page 46 in the A.I.S.I. Steel Products Manual for carbon steel wire and rods May 1957. Fractional gages are produced with zinc coatings equivalent to those of next smaller size, which in this case is 7 gage, and 7 gage specifies that 0.40 oz. per sq. ft. is standard.

If both sizes were produced with exactly 0.40 oz. of coating both would have 0.0068 inches in thickness. The measurement of a zinc coating is not usually considered in specification writing because its measurement is much too delicate for production work. Because of the iron-zinc alloy found in hot dipping this interferes with the actual zinc value. The figure stated above is a corrected figure for hot dipping practice.

Cut ends of galvanized wire do bleed over and form a protection when the steel has been exposed.

I trust that this letter is a help to you.

L. R. Dillenbeck, Supt. of Metallurgy

Mortar Mix Specifications for Unit Masonry

SECTION 12

In connection with the use of proper reinforcement, the proper use and formulation of mortar is of prime importance in the design and construction of sound, crack-free masonry walls. The following mortar mix data combines material to be found in A.I.A. File No. 3-A with suggestions received from the Portland Cement Association. Many thanks are due J. J. Shideler, Manager, Products and Applications Development Section of the association, for reviewing this data.

Scope:

This division of the specifications covers mortars for all types of masonry required to complete the masonry work indicated on the drawings and in the specifications. Mixing of the mortar and its physical requirements are described.

Materials:

- **a.** Masonry Cement (Type II): ASTM Designation C91
- **b.** Portland Cement (Types I, II or III): ASTM Designation C150
- **c.** Air-Entraining Portland Cement: ASTM Designation C175
- **d.** *Hydrated Lime: ASTM Designation C207, type S
- **e.** *Aggregates: ASTM Designation C144
- **f.** Water: Water shall be clean and free from deleterious amounts of acids, alkalies or organic materials.
- **g.** Admixtures or Mortar Colors: If admixtures (water repellents, plasticizing agents, etc.), or mortar colors are to be added to the mortar at time of mixing, they shall be provided for in the contract specifications.
- **h.** **Anti-Freeze Compounds: No anti-freeze liquid, salts or other substances shall be used in the mortar to lower the freezing point.
- **i.** Air-Entraining Agents: Air entraining agents used in cement-lime mixes shall meet ASTM Designation C260. The use of air entrainment in cement-lime mixes is a moot question. Bond strengths are reduced as much as 25 per cent by 12 per cent entrainment of air.

Property Specifications:

Mortar shall consist of a mixture of cementitious material and aggregate together with the following additional requirements:

- **a.** *Water Retention:* Mortar of the materials and proportions to be used in the construction, mixed to an initial flow at 105 to 115, shall have a flow after suction of not less than 80%.
- **b.** *Compressive Strength:* Mortars made of the materials and in the proportions to be used in the construction shall be classified by the following minimum compressive strengths (psi) of three 2-inch cubes at 28 days:

Type M	2500 psi (See par. 5a)
Type S	1800 psi (See par. 5a)
Type N	750 psi (See par. 5a)

  Compressive strength shall be determined in accordance with ASTM specification C270.

- **c.** *Proportions:* The damp loose volume of aggregate in the mortar shall not be less than 2¼ nor more than 3½ times the total amount of cementitious materials used.
- **d.** *Soundness:* The soundness test for masonry cement in ASTM Designation C91 shall be applied to all cementitious materials as combined in the job proportions.

*Weight of lime considered to be 40 lb./cu ft. and 1 cu. ft. of damp loose sand to contain 80 lb. of dry sand.

**Since some salts and chemicals used in anti-freeze compounds may have a deteriorating effect on steel, their use in mortars when joint reinforcing is used should definitely be avoided.

Proportion Specifications:

Unless the strength classification of the mortar has been established by tests in accordance with the property specification section, mortars using the following cementitious materials may be assumed to meet the compressive strength classification shown when mixed with aggregate in the proportion recommended.

Classification and Proportions:

Type M 1 Masonry Cement, 1 Portland Cement, not less than 4½ Sand nor more than 6 Sand

or 1 Portland Cement, ¼ Hydrated Lime, not less than 3 Sand nor more than 3¾ Sand

Type S 1 Masonry Cement, ½ Portland Cement, not less than 3½ Sand nor more than 4½ Sand

or 1 Portland Cement, ½ Hydrated Lime, not less than 3½ Sand nor more than 4½ Sand

Type N 1 Masonry Cement, not less than 2¼ Sand nor more than 3 sand.

or 1 Portland Cement, 1 Hydrated Lime, not less than 4½ Sand nor more than 6 Sand

Mixing:

a. *Machine mixing:* About half the water shall be placed in the mixer followed by one-half the sand, the cementitious materials, the rest of the sand, and the balance of the water to obtain the desired consistency. The mixer shall be operating during the addition of the materials and shall be operated for at least five minutes after all materials are in, to assure that the mortar is thoroughly mixed.

b. *Hand mixing:* Hand mixing of the mortar may be permitted on small projects with the written approval of the architect. The mixing shall take place in a water-tight mixing box. The cementitious materials shall be mixed with the aggregate 4 or 5 times or until the mixture has a uniform color. Water then shall be added slowly to the desired consistency with continued mixing during the addition. The maximum amount of water shall be used that is consistent with good workmanship, but under no condition shall water be added in such quantities to cause flooding. If an air-entraining agent is used, it shall be added to the water.

Let the fresh mortar, whether machine-mixed or hand-mixed, remain in a box until the mixture has had a chance to take up all the water. If necessary, water shall be added to produce the desired workability.

c. *Retempering:* Mortar which has stiffened because of chemical reactions associated with hardening caused by initial set of cement in the mortar shall not be used. However, mortar which has stiffened because of the loss of mixing water through evaporation and absorption shall be retempered to restore its workability.

Storage of Materials:

Cementitious materials and aggregates shall be stored in such a manner as to prevent deterioration or intrusion of foreign material. Any material which has become unsuitable for good construction shall not be used.

Cost of Tests:

The costs of any required test shall be borne by the contractor. Manufacturer's mill test may be acceptable.

SECTION

13

Supplementary Technical Background Data

Introductory Comment:

In the following pages we give you a basic roundup of pertinent excerpts from the findings, and recommendations developed and published by various authoritative bodies (O.C.E., B.R.A.B., N.C.M.A.) concerned with sound masonry wall construction. While this data does not coincide in all respects with Dur-o-wal organization's own research, it does nevertheless constitute an "official" frame of reference invaluable to anyone doing practical work in the masonry field.

Supplementary
Technical
Background Data

O.C.E. DATA

OFFICE OF CHIEF OF ENGINEERS, CORPS OF ENGINEERS

From Guide Specification for Military and Civil Works Construction—CE-206.01 Nov. 30, 1958.
Only Certain Sections Pertaining to Masonry Wall Design

CONCRETE MASONRY:

Concrete masonry units shall be free of any deleterious matter that will stain plaster or corrode metal. Units that have been subjected during manufacture to a saturated steam pressure of not less than 120 pounds per square inch for 5 hours or more may be used as soon as cooled. Units that have not been subjected to such steam pressure shall be cured for 28 days or more before delivery. Units shall be delivered to the job site in an air-dry condition, and shall be classified into group 1 or group 2 depending upon the linear shrinkage potential as stipulated in Table II.

Table II
LINEAR SHRINKAGE POTENTIAL

Concrete density, lbs. per cu. ft.	Maximum linear shrinkage percent (ASTM Standard C 426)	
	Group 1	Group 2
120 or more	0.065	0.03
Less than 120	.065	.04

NOTE: This table applies to concrete masonry units other than concrete brick and split block which the specification covers in a separate table not reproduced herein.

FROM NOTES TO CONTRACTING OFFICER

Note 19:——— For concrete masonry the locations and details of bond beams, control joints, and joint reinforcement for shrinkage-cracking control will be shown on contract drawings. The procedures and criteria for crack-control design are indicated in Notes 20, 21 and 22 for walls of concrete masonry units and for concrete brick and split block with masonry backup.

Note 20:———Bond beams exert a restraining influence on the two or three adjacent courses and therefore are considered to be effective in controlling shrinkage cracking. Bond beams will be cut at stated intervals as given in Table III. Bond beams will be in the same course of masonry between bond-beam breaks and will be continuous between control joints, but control joints will be located at all breaks in the bond beam. Continuous lengths of a bond beam will not exceed the lengths given in Table III.

Table III
MAXIMUM LENGTH OF BOND BEAMS OR CONTINUOUS LENGTHS OF JOINT REINFORCING—FEET

Units	Exterior walls	Partitions
Group 1	40	50
Group 2	50	60

In walls with no openings between control joints, bond beams will be provided at intervals of not more than 4 feet throughout the height of the wall.

In walls with openings, bond beams will be provided immediately below the sill and at or immediately above the lintel. In some structural designs a bond beam located at or immediately above the lintel may be made to fulfill the requirement of a bond beam at the top of the wall. A bond beam will not be required between openings where the relationship of the height of the opening to the distance between the openings is not greater than that given in the joint-reinforcing columns marked None, in Tables IV and V.

Table IV
CONTROL JOINTS AND JOINT REINFORCING—EXTERIOR WALLS (FACINGS OF CONCRETE-BRICK, SPLIT-BLOCK, AND CONCRETE MASONRY)

Control-joint spacing—feet						
Wall height (feet)	Group 1 units					
	Joint reinforcing					
	None		16″ on center		8″ on center	
	Av.	Max.	Av.	Max.	Av.	Max.
To 8	15	20	20	25	25	30
8 to 12	20	25	25	30	30	35
Over 12	25	30	30	35	35	40
Wall height (feet)	Group 2 units					
	Joint reinforcing					
	None		16″ on center		8″ on center	
	Av.	Max.	Av.	Max.	Av.	Max.
To 8	20	25	30	35	35	40
8 to 12	25	30	35	40	40	45
Over 12	30	35	40	45	45	50

Table V
CONTROL JOINTS AND JOINT REINFORCING FOR PARTITIONS

Control-joint spacing—feet						
Wall height (feet)	Group 1 units					
	Joint reinforcing					
	None		16″ on center		8″ on center	
	Av.	Max.	Av.	Max.	Av.	Max.
To 8	15	20	25	30	35	40
8 to 12	20	25	30	35	40	45
Over 12	25	30	35	40	45	50
Wall height (feet)	Group 2 units					
	Joint reinforcing					
	None		16″ on center		8″ on center	
	Av.	Max.	Av.	Max.	Av.	Max.
To 8	25	30	35	40	45	50
8 to 12	30	35	40	45	50	55
Over 12	35	40	45	50	55	60

Table VI
CONTROL JOINTS AND JOINT REINFORCING FOR VENEER CONSTRUCTION (FACINGS OF CONCRETE BRICK, SPLIT BLOCK, AND CONCRETE MASONRY)

Control-joint spacing—feet					
	Group 1 units				
	Joint reinforcing				
Wall height (feet)	None	16″ on center		8″ on center	
		Av.	Max.	Av.	Max.
To 8	10	20	25	25	30
8 to 12	15	25	30	30	35
Over 12	20	30	35	35	40
	Group 2 units				
	Joint reinforcing				
Wall height (feet)	None	16″ on center		8″ on center	
		Av.	Max.	Av.	Max.
To 8	15	35	40	45	50
8 to 12	20	40	45	50	55
Over 12	25	45	50	55	60

"Av."—Average length of wall panels into which walls are subdivided by control joints.

"Max."—Maximum length of wall panels into which walls are subdivided by control joints.

For load-bearing walls, both interior and exterior, a bond beam will be provided between the lintel level and the top of the wall in one-story buildings, and at ceiling levels in multistory buildings.

For non-load-bearing walls, both exterior and interior, a bond beam will be provided as indicated above for load-bearing walls; or, when installation of a bond beam is not practical for structural reasons, either of the following acceptable substitutes will be provided:

a. Three courses of joint reinforcing immediately below the top of the wall. Continuous lengths of joint reinforcing will not exceed the lengths given in Table III above.

b. Control joints as indicated in Table VII.

Parapets will be provided with crack-control measures comparable to those of non-load-bearing walls.

Table VII
MAXIMUM CONTROL-JOINT SPACING REQUIREMENTS—FEET (NON-LOAD-BEARING WALLS AND VENEER CONSTRUCTION WITHOUT BOND BEAMS OR JOINT REINFORCING)

Wall height (feet)	Group 1 units	Group 2 units
To 8	10	15
8 to 12	15	20
Over 12	20	25

Structural elements, such as poured foundation walls, spandrels, and concrete floors and roofs that are bonded to the masonry and reinforced near the bond plane with bars extending parallel to the wall length, exert, in the same manner as bond beams, a restraining influence on masonry courses.

Note 21:———Joint reinforcement is one of the means of preventing conspicuous cracking. Joint reinforcing need not be located closer to a bond beam than three 8-inch-block courses from the beam. Joint reinforcing will be interrupted at control joints.

For veneer construction, three courses of joint reinforcing 8 inches apart, measuring vertically, will be provided in lieu of bond beams at the top of the wall in one-story buildings, and at ceiling levels in multi-story buildings. Continuous lengths of joint reinforcing will not exceed the lengths given in Table III.

Wall panels will not exceed the length given in Table III. A wall panel as referred to here is an element or section of wall in one plane and extending between the extremities of a wall, or from an end of a wall to a control joint, or between two control joints.

Note 22:———Control joints will be located where stress concentrates are the greatest, such as at jambs of openings, at pipe and column chases, at bond-beam breaks, at abutment of columns and walls, at locations where changes in wall height or wall thickness occur, and over weak planes in foundation walls. The contact drawings will include details of control-joint extensions through bond beams at intervals not exceeding those shown in Table III.

Control joints and joint reinforcing may be used in combination for shrinkage-cracking control. When this method is used control joints and reinforcing will be provided as indicated in Tables IV, V, and VI.

Control joints spaced as shown in Table VII may be provided at the top of non-load-bearing walls and of veneer construction in lieu of bond beam or of three courses of joint reinforcement.

The combination control-joint and joint-reinforcing method, but not the bond-beam method, may be used in veneer construction.

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In walls of group 1 or group 2 units, Tables IV, V, and VI, and with openings between control joints, either additional control joints will be provided at jambs of openings or reinforcing will be provided in the two bed joints immediately below the sill and above the lintel. The reinforcing will extend 24 inches beyond both sides of the openings. If control joints are used, only one control joint will be used for openings less than 6 feet wide, and this control joint will be placed at one jamb of the opening.

No joint reinforcing or control joint will be required between openings where the relationship of height of the opening to the distance between the openings is not greater than that given in Tables IV, V, and VI in the joint reinforcing columns marked None.

COMMENTS BY DUR-O-WAL, O.C.E. DATA:

You will note under Note 20, that bond beams exert a restraining influence on 2 or 3 courses adjacent to them, which would bear out the original Bureau of Standards Report that more effective reinforcing can be accomplished by joint reinforcement because of better distribution of steel.

We realize that the height of the wall has a bearing on the spacing of control joints and resistance of wall to cracking. The engineers have attempted to take this into consideration in recommending their spacing of control joints when used in combination with joint reinforcement.

You will also notice in Table II that they refer to Group 1 and Group 2 units. Generally, Group 1 masonry units are those normally manufactured and available by low pressure or regular steam curing methods. Most of the available concrete masonry used today is cured in this manner and should be the type of masonry of most concern to the construction industry. Also, generally, Group 2 units are those cured by the autoclaving or high pressure method. This method of curing reduces the shrinkage characteristics somewhat, and you will notice on the tables that the spacing of control joints is somewhat farther apart than for the Group 1 units.

The principal objection to the control joint method when used by itself, is that the walls would be loaded with control joints at all openings and at very close spacing. This leads to unsightly design and is objectionable to many architects. It also is a source of continual excessive maintenance.

The principal objection to the bond beam method is that it is costly because the masonry work is continually interrupted and must be stopped at this point to permit the placing of concrete for the bond beams. Masonry contractors find that when masonry work must be kept level, the slower or less productive masons control the speed of the work. Also, it is the source of possible water accumulation and leakage.

B.R.A.B. DATA

BUILDING RESEARCH ADVISORY BOARD

Following is a reprint (reproduction) of pp. 10-14 of NAS-NRC Publication 447, "Cracking of Concrete Face Brick and the Development of Data Necessary for the Establishment of Criteria for its Manufacture and Installation," prepared by a Special Advisory Committee of the Building Research Advisory Board for the Federal Housing Administration.

The full report may be obtained through Printing and Publishing Office, National Academy of Sciences — National Research Council, 2101 Constitution Avenue, Washington 25, D. C. ($1.50)

Table 1

REQUIREMENTS FOR PHYSICAL PROPERTIES OF FACE BRICK TO MINIMIZE WALL CRACKING

Weight of Brick Units (lbs. per cu. ft.)	Min. Comp. Strength Brick "Flatwise" (psi, avg. gross area)[1]	Max. Water Absorption (lbs. per cu. ft.)	Max. Linear Shrinkage Allowance (percent)[2]	
			Group I[3]	Group II[4]
120 or more	3500	9	0.025	—
85 to 120	2500	12	0.035	—
Less than 85	2500	15	0.035	—

Note: All of the above brick are to be in an air-dry condition as defined in 6.1, p. 00.

The above table should be enlarged to include tensile strength requirements when available (see research recommendations, p. 00).

[1] Average of 5 brick. Strength of individual brick should not be less than 80 per cent of the average strength.

[2] Group I is the preferred classification.

[3] These values were arrived at using the British Method of test as defined in the Journal of the American Concrete Institute, Vol. 24, No. 28, ACI Report 716, Paper No. 49-53, April 1953, pp. 746-747.

Linear shrinkage, to obtain qualifying values for this group, should be determined by the British Method.

[4] Face brick which comply with the strength and absorption requirements but which have not been tested for shrinkage, or may not meet the shrinkage allowances required for Group I brick, are classed as Group II.

13.0 Control Joints and Horizontal Reinforcement

The use of control joints[1] and horizontal joint reinforcing depends largely upon the stability of the brick used and the length of a wall in a single horizontal run. A certain amount of reinforcement is desirable in all cases and necessary in the case of stacked bond. Table 2 suggests general requirements for control joints and horizontal reinforcement for concrete face brick walls. (See also 13.1)

Table 2

GENERAL REQUIREMENTS FOR SPACING OF CONTROL JOINTS AND HORIZONTAL JOINT REINFORCEMENT FOR CONCRETE FACE BRICK WALLS

Air-Dry[1] Brick	Average Control Joint Spacing in Feet		
	No. Reinforcement[4]	Reinforcement[5] Every 6th Joint (16 in. o.c. vert.)	Reinforcement[5] Every 3rd Joint (8 in. o.c. vert.)
Group I[2]	24 ft.	30 ft.	36 ft.
Group II[3]	16 ft.	20 ft.	24 ft.

Note: The maximum spacing between 2 control joints should not exceed the average spacing of all joints in any one wall by more than 25 per cent.

[1] All face brick are to be in an air-dry condition as defined in 6.1, p. 11.

[2] As defined in Table 1, p. 10.

[3] As defined in footnote 4 of Table 1, p. 10.

[4] Horizontal reinforcement should be used at all openings and with stacked bond as outlined in 13.3 and 13.4, respectively.

[5] Joint Reinforcement as defined in 13.2.

13.1 Reduced Requirements for One-Story Walls Less than 48 Feet Long[1]

Due to less rigid restraint with short one-story walls the following concessions may be made for control joints and horizontal reinforcement for single-story walls less than 48 ft. long.

With Group I brick neither control joints nor joint reinforcement should be required.

With Group II brick either control joints at an average spacing of 20 ft. or joint reinforcement every 16 in. should be required.

Both control joints at 20 ft. spacing and joint reinforcing every 8 in. should be required for brick which do not conform to requirements outlined in Table I, p. 10 for brick in Groups I and II. However, these brick are to be in an air-dry condition as defined in 6.1, p. 11 and other requirements for Grade A brick of A.S.T.M. Designation C 55, *Standard Specifications for Concrete Building Brick.*

One member of the Committee, Mr. Benjamin Wilk, believes that these requirements as they apply to Group II brick are excessive and unduly penalize those brick not meeting the linear shrinkage allowances established for the Group I classification.

13.2 Joint Reinforcement[1]

Horizontal joint reinforcement, where required, should consist of not less than two deformed longitudinal No. 9 gage (or heavier) high elastic limit cold drawn steel wires having a tensile strength of 80,000 psi when tested across a weld after deforming and welding. Cross wires should be ⅛ in. diameter (or heavier) of the same quality.

13.3 Openings

Horizontal joint reinforcement should be provided in the first and second bed joints immediately above and below all openings and should extend 24 in. beyond the opening. This requirement is to be general for all conditions outlined in 13.0 and 13.1.

13.4 Stacked Bond

In all cases stacked bond, whether flat or on end, should be reinforced horizontally at no more than 16 in. vertical intervals for the full length of the wall.

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[1] A control joint is a continuous vertical separation in the face brick wall which, by limiting the length of the wall, limits the potential expansion and contraction.

[1] Horizontal reinforcement shall be used at all openings and with stacked bond as outlined in 13.3 and 13.4, respectively. See also research recommendation 2.3, p. 5.

OTHER STRUCTURAL CONSIDERATIONS

It is recognized that structural weaknesses, including settlement of foundations, deflection of footings, and movement of "back-up" will contribute to the cracking of concrete face brick walls. It is not within the scope of the work of this Committee to do other than call attention to the fact that where concrete brick is used as a facing material, the structural design of footings, foundations, "back-up," etc., should be such as to minimize the introduction of extraneous stresses into the face brick wall.

COMMENTS BY DUR-O-WAL, B.R.A.B. DATA:

During 1955 and 1956 the Building Research Advisory Board conducted a study for the Federal Housing Administration on the subject "Cracking of Concrete Face Brick and the Development of Data Necessary for the Establishment of Criteria for its Manufacture and Installation." The report, issued May 11, 1956, deals with specifications for masonry and general requirements for spacing of control joints and horizontal joint reinforcement in concrete face brick walls. In this recommendation the requirements for joint reinforcement are not as involved as those outlined by the U. S. Engineer Corps. They are excellent recommendations, however, and indicate another approach to the problem.

The difference in the two reports is in the classification of masonry. This is important and should be carefully noted when you compare the two recommendations. The Army Engineers list the most readily available regular, low steam cured block as Group I and the preferred autoclave, low shrinkage block as Group 2. The Building Research Advisory Board recommendation reverses this procedure.

Recommendations for use of Joint Reinforcement in Masonry Walls as noted in N.C.M.A. Design Manual for Concrete Masonry Foundation Walls

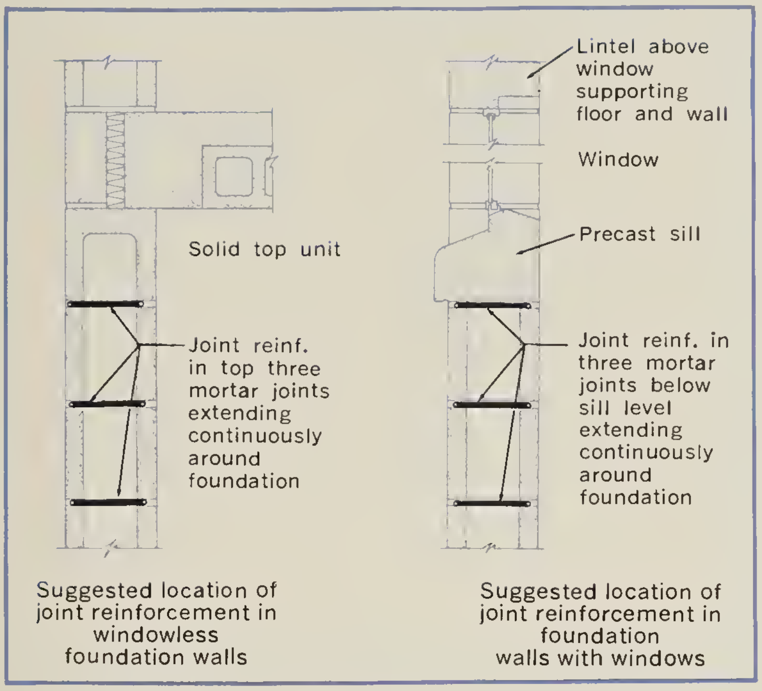

Lintel above window supporting floor and wall
Window
Precast sill
Solid top unit
Joint reinf. in top three mortar joints extending continuously around foundation
Joint reinf. in three mortar joints below sill level extending continuously around foundation
Suggested location of joint reinforcement in windowless foundation walls
Suggested location of joint reinforcement in foundation walls with windows

Figure 21—Types and Suggested Location of Horizontal Joint Reinforcement in Concrete Masonry Foundation Walls.

Referring to Fig. 21, the approximate ultimate resisting moment of a cracked section reinforced with joint reinforcing may be calculated from the following formula:

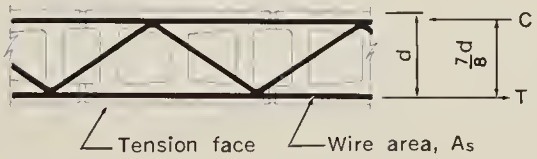
$$M_s = \frac{7}{8} A_s f_y d, \text{ where,}$$

A_s = effective cross-sectional area of wire embedded in mortar on tension side of the section, sq. in.

f_y = yield point strength of wire, psi (64,000 psi for cold-drawn steel wire)

d = depth of section from center of tension wire to compression face of section, in.

To develop its yield point resistance, the wire must be strongly bonded to the mortar and the mortar must be well bonded to the masonry units.



Ultimate resisting moment, $M_s = \frac{7}{8} A_s f_y d$

Figure 21—Horizontal Section at Mortar Joint Containing Embedded Joint Reinforcement.

A deformed type of wire and a strong mortar, as specified elsewhere, are recommended.

Calculated ultimate resisting moments for 8, 10, and 12 in. thick walls containing various amounts of joint reinforcement are given in Table 12 on this page. The values are based on the assumption of a cracked section and on the minimum yield point strength of cold-drawn steel wire (64,000 psi), and are conservative with respect to the actual ultimate moment which would be required to cause actual collapse or total failure. When comparing the calculated values given in Table 12 with actual or calculated ultimate resisting moments for unreinforced walls, as given in Table 6, page 38, recognition should be given to the inherent reserve strength and the greater reliability of reinforced as compared to unreinforced walls. This may be accomplished by varying the safety factor employed in arriving at design resisting moment values for the respective wall constructions. For example, the values given in Table 12 may be divided by two to arrive at a design resisting moment for a reinforced wall whereas the design resisting moment for an unreinforced wall may be taken as one-quarter of the ultimate values given in Table 6, or some lesser or greater value depending upon the strength of the units and the probability of vertical cracking developing in and impairing the horizontal flexural resistance of the unreinforced wall.

Where it is desired to employ stacked bond construction in basement walls, it is recommended that continuous horizontal reinforcement, sufficient to provide a horizontal flexural resistance at least equal to that which would be obtained with

Table 12

DESIGN ULTIMATE (YIELD POINT) RESISTING MOMENTS IN THE HORIZONTAL SPAN FOR CONCRETE MASONRY WALLS REINFORCED WITH JOINT REINFORCEMENT (WALLS BUILT WITH TYPE M OR S MORTARS)

Nominal Wall Thickness Inches	Joint Reinforcement Spacing, Inches	Ultimate Resisting Moment, * in lbs. per ft. of wall height		
		Longitudinal Wire Size		
		No. 9	No. 8	³⁄₁₆ in.
8	24	3,240	3,940	5,270
	16	4,890	5,920	7,900
	8	9,740	11,800	15,800
10	24	4,200	5,080	6,800
	16	6,310	7,650	10,200
	8	12,600	15,250	20,400
12	24	5,150	6,240	8,360
	16	7,750	9,400	12,550
	8	15,450	18,750	25,100

*Assuming use of prefabricated joint reinforcement made from cold-drawn steel wire and with parallel side rods spaced 6 in., 8 in. and 10 in. apart for the 8 in., 10 in. and 12 in. walls respectively. A yield point strength of 64,000 psi and effective depths of 6.82 in., 8.82 in. and 10.82 in. were assumed in the calculations. Resisting moments based on ultimate strength of wire (80,000 to 100,000 psi) will exceed the values shown by 25 to 56 per cent.

running bond, be incorporated in the wall construction. As a general rule, bond beams reinforced with 2 No. 5 bars (one near each face) and spaced not more than 42 in. o.c., or joint reinforcement with No. 9 gage or larger side wires in each horizontal mortar joint 8 in. o.c., will be adequate.

When considering the use of bond beams and joint reinforcement in foundation walls, it is important to remember that neither of these measures will add appreciably to the strength of the walls in the vertical span. Also, the increase in horizontal flexural strength which can be obtained through use of joint reinforcement is limited, due to practical limitations on the amount of steel which can be embedded in the joints and the bond strength which can be developed between the mortar, the reinforcement, and the wall units. Generally speaking, it will be prudent to neglect any theoretical increase in horizontal flexural strength which may be obtained with joint reinforcement and to base the design of walls on the same wall height, depth below grade and lateral support requirements recommended for unreinforced walls. Bond beams, on the other hand, may be so designed as to contribute appreciably to wall strength and stiffness and the theoretical increase in these values may usually be taken into account in the design.

7. Expansion and Control Joints

It will seldom be necessary to provide expansion or control joints in concrete masonry foundation walls. The range of thermal and moisture volume change occurring in such walls will generally be quite low due to the insulating effect and protection against rapid or excessive drying afforded by the earth in contact with the walls. As a result, control joints seldom, if ever, are recommended in foundation walls under about 100 ft. in length. In longer walls, control joints at spacings of 35 to 40 ft. may be desirable in some cases, particularly where a major portion of the wall height will project and be exposed above grade. Ordinarily, expansion joints are not recommended in foundation walls except where required by, and aligned with, expansion joints in the superstructure.

Table 6
FLEXURAL STRENGTH OF PLAIN CONCRETE MASONRY WALLS IN THE HORIZONTAL SPAN*

Wall Construction †					Ultimate Resisting Moment, in. lb./ft.	Modulus of Rupture, psi	
No. of Wall Panels Tested	Wall Thickness, inches	Units		Mortar Compressive Strength, psi		Based on Assumed Solid Section	Based on Actual Hollow Section
		Condition When Laid	Compressive Strength, psi (gross area)				
3	4	Air-Dry	1370	500	8,380	320	350
9	8	Air-dry	1720	2740	18,930	160	225
				500	19,100	165	230
				290	17,200	145	205
9	8	Damp	1720	2740	17,800	150	210
				500	17,600	150	210
				290	19,500	170	235
3	12	Air-dry	1330	500	21,100	80	135

* Data from tests conducted at Iowa State College for the National Concrete Masonry Association. For complete information, see "Flexural Strength of Hollow Unit Concrete Masonry Walls in the Horizontal Span," NCMA, 1958.

† All walls (4 ft. high by 6 ft. 8 in. long) were built with three-core hollow expanded shale aggregate units laid in running bond with face shell mortar bedding. Tests were conducted at 28 days with walls in the vertical position, simply supported at the sides, free at the top and bottom edges, and with an applied compressive load of 10 psi on the gross area in the vertical direction. Test load applied at the midpoint of the horizontal span.

SPECIAL NOTE: Modulus of rupture and resisting moment values shown in the above table will not be similar for all block walls. The horizontal flexural strength of plain walls will vary depending upon the tensile strength, compressive strength, and design of the units and other factors.

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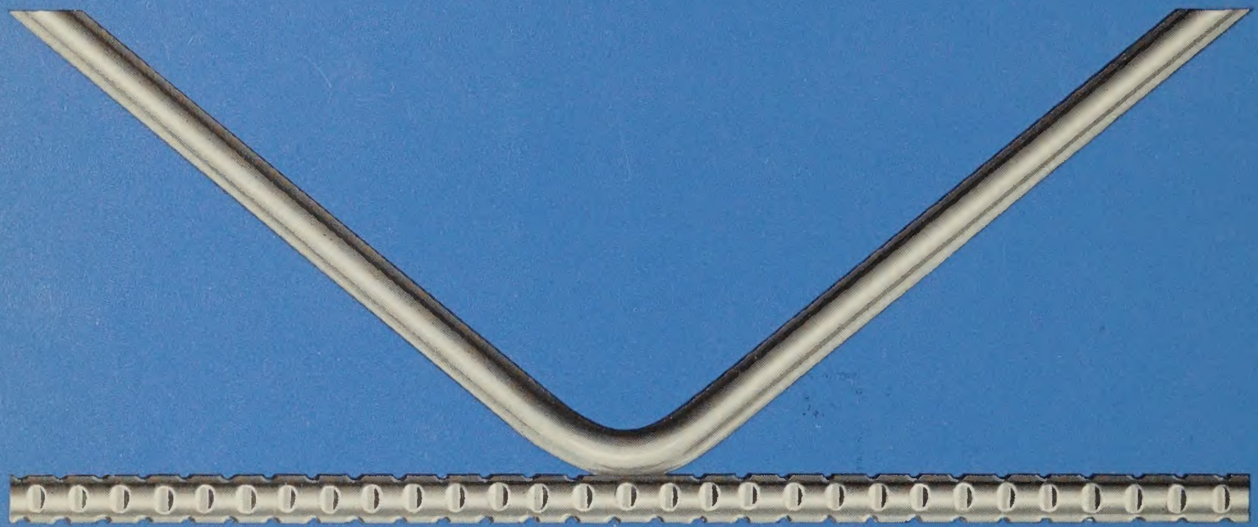

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